# Identification and testing of measures to improve motorway work zone safety

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*"For every complex problem there*
*is an answer that is clear, simple,*
*and wrong".*
*H. L. Mencken*

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# Abstract

Roadway work zones are hazardous, for both workers and motorists who drive through the complex array of signs, delineators and lane changes. Improper lane changing manoeuvres and possible vehicle encroachments in the activity areas may cause injuries to both the car occupants and road workers.

Several studies agree that the presence of work zones significantly increases the risk of road crashes. Excessive speeding and high speed variances have also been identified as major causes of a large percentage of work zones crashes, injuries or fatalities.

Although most work zones are controlled by reduced speed limits or state law enforcement, the driver's compliance with these regulations is still minimal.

The main objective of this research was to determine safe and effective countermeasures for the reduction of speeds and speed variances within work zones. Furthermore, the influence of work zone layout features on crash occurrences was another important issue addressed with this study.

An extensive accident analysis was therefore conducted on the stationary work zones of the Italian motorway network in order to identify the most critical layouts in terms of safety. The Empirical Bayes (EB) before-and-after method was performed in order to evaluate the change in the expected crash frequencies associated with the installation of work zones on motorways. A dataset of 15,570 stationary work zones including crash data, road segment data and traffic data was analyzed in order to estimate crash modification factors (CMFs) associated with the different layout configurations. The findings of this part of the research have shown that all layout configurations that involve a crossover were associated with the highest values of CMFs.

A typical motorway crossover, designed in accordance with the Italian Ministerial Decree 10 July 2002, was then defined and implemented at the driving simulator of the University of Florence (Italy).

A number of countermeasures have been tested in virtual reality through nine different configurations of the work zone crossover, in order to evaluate their effectiveness in reducing speeds and speed variances. The experiments investigated the effects of different speed limit sequences and alternative design features, such as wider

lanes and median openings. Furthermore, the effects of different channelizing devices and perceptual treatments based on the Human Factor (HF) principles have also been investigated.

The results of the experiments, performed on a total sample of 42 subjects, showed that, for all configurations, the drivers' speeds are always higher than the posted speed limits and decrease significantly only in approaching the crossover by-passes. The implementation of higher speed limits, together with a wider median opening, led to a greater homogeneity of the driving speed. Perceptual countermeasures generally induced both the greatest homogenization of speeds and the largest reductions in mean speed values.

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# Chapter 1.

# Introduction

## 1.1 Overview

Work zones are critical sections of the road network in terms of safety, as drivers have to face additional choices because of the temporary and unfamiliar road layout. Roadway work zones are hazardous, for both workers and motorists who drive through the complex array of signs, delineators and lane changes. Improper lane changing manoeuvres may cause additional conflict points between vehicles. Furthermore, possible vehicle encroachments in the activity areas can cause injury to both the car occupants and road workers.

According to the latest available statistics, 669 work zone fatalities occurred in the U.S. in 2014, accounting for 2% of all roadway fatalities (FHWA, 2015). According to FHWA data, out of a total of 87,606 work zone crashes occurred in the U.S. in 2010 (1.6% of the total number of roadway accidents), only 514 (0.6%) were fatal crashes, resulting in 586 fatalities. Injury and property damage only (PDO) crash rates were respectively 30% and 69%.

During the period 2003-2012, 2,435 severe crashes occurred in proximity of roadworks in Sweden (Liljegren, 2014). Of the 2,435 accidents, 42 (1.7%) were fatal accidents and 520 (21.4%) were accidents with serious injuries.

In Italy, 762 work zone crashes, with fatalities or injuries, occurred within the 3,000 km long motorways managed by Autostrade per l'Italia S.p.A. (ASPI) during a 6-year period from 2007 to 2012. Such accidents resulted in 21 fatalities (in 16 crashes) and 1,252 injuries (in 746 crashes).

Work zone crashes can be organized into two categories (Mohan and Zech, 2005):

1. drivers-related crashes: work zone accidents where the driver's incursion or collision, in the activity area or in proximity of the work zone, with workers,

work zone equipment or other vehicles results in injuries or fatalities of workers and/or drivers;

2. occupational crashes: work zone accidents where a worker suffers injury or death due to work equipment or environmental factors within the activity area.

Statistics indicate that about 80% of fatalities related to work zone presence involve motorists (FHWA, 2015). According to the National Safety Council, over 100 road construction workers are killed in the U.S. each year. Nearly half of these workers are killed as a result of being struck by motor vehicles. This data justifies the efforts made on regulating motorist driving behaviour within work zone areas.

The identification of the characteristics and the major causes of work zone crashes is a critical step towards developing effective safety countermeasures for roadworks. A better understanding of vehicle and driver interactions with work zone features will consequently prevent these crashes from occurring, and could help achieve better traffic efficiency.

Excessive traffic speeds is a major safety concern in work zones and a potential risk for both motorists and workers (Bryden et al., 2000; Dissanayake and Akepati, 2009; Garber and Zhao, 2002; Li and Bai, 2007; Li and Bai, 2009).

On the other hand, several studies have shown that a large speed variance coupled with hazardous conditions (e.g., worker presence) could also increase work zone crash frequencies (Garber and Gadiraju, 1998; Migletz et al., 1998; Salem et al., 2006). Crash statistics identified rear-end collisions as being the most common crash type in work zone areas (Bay and Li, 2006; Garber and Zhao, 2002; Ullman et al., 2008). The predominance of rear-end collisions strongly suggests that speed variance is a major cause of work zone crashes.

The implementation of countermeasures that reduce speed variance, or that cause drivers to drive at approximately the same speed throughout a work zone, are needed in order to increase safety. A wide variety of traffic calming methods have also been used to reduce speed and speed variance in work zones (ASAP, 2013). Police enforcement is however considered to be the most effective method of improving speed limit compliance, but only when police presence is connected to continuous and on-going enforcement activities. Other promising devices are those related to various message signs and speed monitoring techniques, where drivers are provided with real time information on their current speed. Nevertheless, these methods have limits in terms of effectiveness and the effect is often localized in the proximity of the device or tends to decrease in the long-term.

Section 6B.01 of the Manual on Uniform Traffic Control Devices (MUTCD) specifically states that temporary traffic controls at work sites, should be designed considering the assumption that drivers will only reduce their speed, if they clearly perceive a significant life-threatening risk (FHWA, 2009).

An appropriate work zone design is therefore a major priority, in order to increase safety for both workers and motorists, who drive through the complex array of signs, barrels and lane changes. The use of design elements, that meet driver expectations and that avoid unexpected geometric features, together with an appropriate speed management strategy could help moderate speeds and speed variances and therefore provide safer driving conditions.

## 1.2 Research objectives

In order to investigate and enhance current and future work zone safety during work zone operations, this study focuses on the following research objectives:

1. provide a better understanding of the contributing factors that cause injury and fatal crashes within work zones, and of the most effective speed management method through work zones;
2. investigate and quantify the impact of motorway work zone layout parameters, on risk crash occurrences;
3. study speed behaviour in response to different signalling schemes and different work zone layout features, and evaluate the effectiveness of different countermeasures in managing speed within the work zone areas.

The first aim was to conduct a thorough review on the literature regarding work zone crashes, interventions and eventual research gaps. This study is provided in the background section (Chapter 2). From the literature analysis, the potential work zone parameters and the contributing causes of zone crashes were investigated. Speeding was a major factor in work zone fatalities, and a number of speed management strategies were identified to address this issue.

Perceptual countermeasures appeared to be one of most promising methods, designed to reduce travel speeds by influencing speed perception, mental workload and risk perception. These treatments offer a low-cost solution in reducing travel speed at hazardous locations, even though their effectiveness may be, to some degree, site specific.

Consequently, an extensive accident analysis on Italian stationary work zones has been conducted in order to identify the most critical work zone layouts in terms of safety. Specifically, the study focuses on quantifying and comparing the impact of different work zone layout configurations (such as lane closure, crossovers, closure of the carriageway) on the expected frequency of severe crashes (resulting in fatalities and/or injuries) through the use of the Empirical Bayes before-after method. This study is reported in Chapter 3.

Finally, the research work aimed to investigate the drivers' speed behaviour within nine different configurations of a work zone crossover in order to identify safer driving measures and conditions. This study, reported in Chapter 4, was implemented in the driving simulator of the Road Safety and Accident Reconstruction Laboratory (LaSIS) of the University of Florence (Italy). It focused on work zone crossovers, as the accident analysis results identified this layout as the most critical for safety. The experiments investigated the speed behaviour in response to different speed limit sequences, geometric features and perceptual treatments.

Specifically, the research aims to regulate motorists' driving behaviour in work zones by:

- lowering mean speeds closer to the temporary speed limits;
- moderating speed variances;
- reducing incidences of sudden increase and decrease in speed.

# Chapter 2.

# Background

## 2.1 Previous studies on work zone crashes

Many studies have been performed on accident experience within work zone areas. This section summarizes the findings of previous studies on work zone crash characteristics in terms of type, location, severity and causal factors.

### *2.1.1 Crash type*

The prevailing types of work zone crashes vary with different locations and times, but most of the previous studies agree that rear-end collisions are the most frequent crash types within work zones (Antonucci et al., 2005; Bay and Li, 2006; Garber and Zhao, 2002; Mohan and Gautam, 2002; Saleh et al., 2013; Ullman et al., 2008)

The study conducted by the University of Florence (Saleh et al., 2013) as a contribution to the ASAP project, a European project addressed to the issues of speed management in work zones (ASAP, 2015), recorded a total of 762 accidents with fatalities or injuries within the 30,389 work zones installed in the Italian motorway in the period 2007-2012. The most frequent type of collision was rear-end (47%) and rear-end, run-off and side collisions were the leading types for fatal and injury work zone crashes in Italy, covering the 74% of the total work zone crashes.

Ullman et al. (2008) investigated a total of 17,228 work zone crashes occurred in California, North Carolina, Ohio, and Washington in a 6-year period from 2000 to 2005. The distribution of crash types by work condition and time of day is presented in Table 2.1. The results showed a significantly different distribution of work zone crash types between daytime and nighttime conditions even though rear-end collision is still the leading type. The proportion of rear-end collisions consistently decreases and a much greater proportion of fixed-object collisions is observed during nighttime, when lower traffic volumes are expected as compared to daytime.

Table 2.1: Percentage of crashes by collision type in daytime and nighttime conditions (Ullman et al., 2008)

| Type of collision | Nighttime work (from 7 p.m. to 6 a.m.) | | Daytime work (from 6 a.m. to 7 a.m.) | |
|---|---|---|---|---|
| | **Active work zone conditions** | **No work conditions** | **Active work zone conditions** | **No work conditions** |
| Rear-End Collision | 33.6% | 26% | 54.4% | 48.7% |
| Sideswipe Collisions | 21% | 15% | 14.8% | 14.8% |
| Fixed-Object Collisions | 21% | 31.9% | 10.3% | 15.9% |
| Other | 24.4% | 25.2% | 20.6% | 14.1% |

German and British studies found that approximately 60% of daytime work zone crashes were rear-end collisions, with the remainder comprised primarily of sideswipes (Dimitropoulos et al., 1998). At night, collisions with fixed objects were of particular concern and were typically associated with inappropriate vehicle speeds.

Other major crash types in work zones include side and head-on collisions (Pigman and Agent, 1990). A study in Georgia found that single-vehicle crashes, side and head-on collisions were the dominant types of fatal work zone crashes (Daniel et al., 2000).

### *2.1.2 Crash location*

A work zone layout can be divided into four areas (Figure 2.1): the advance warning area, the transition area, the activity area and the termination area (FHWA, 2009).

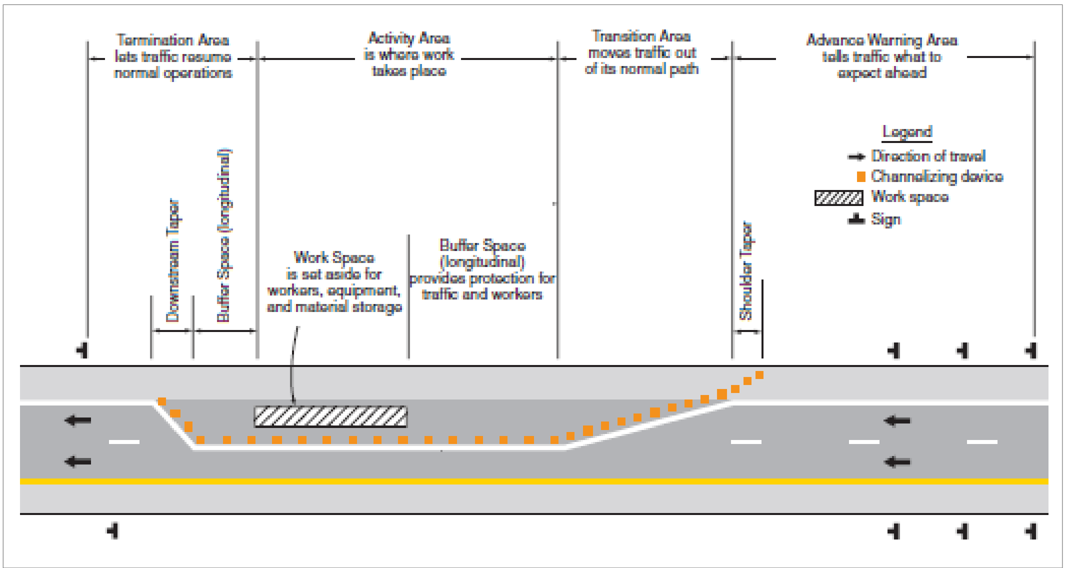



Figure 2.1: Component areas of a work zone (FHWA, 2009)

Previous research did not reach consistent conclusions on the most critical work zone areas. Garber and Zhao (2002) found that rear-end collisions were predominant for all areas except for the termination area, where all crashes were side collisions. Furthermore when traffic moved from the transition area to the activity area, the proportions of rear-end and side collisions decreased and the proportions of fixed-object, off-road, although rear-end crashes were still predominant. Another investigation study points out about 39.1% of crashes occurred in longitudinal buffer area and 16.6% of crashes occurred in the activity area (Nemeth and Migletz, 1978).

Raub et al. (2001) found that the advance warning and transition areas recorded about 40% of all the work zone crashes in the state of Illinois and that greater than 30% of these crashes were injury multi-vehicle crashes.

### *2.1.3 Crash severity*

Inconsistent conclusions have been reached about whether more severe crashes occur in work zones as compared with non-work zone crashes. Despite most studies agree that work zone presence increases the crash frequency, a recent review of the state of the art conducted by Yang et.al (2015) showed that the 48% of previous studies on work zone crashes indicate no clear evidence of the increase in crash severity during work zone conditions. This is also confirmed by the research conducted by Ha and Nemeth (1995) at nine work zones in Ohio which found that work zone crashes were slightly less severe than crashes in “non-work zones” locations.

Some studies from Virginia (Garber and Zhao, 2002), Texas (Ullman and Krammes, 1990) and Ohio (Nemeth and Migletz, 1978) found significant increases of severe crashes in work zones. However other studies (Chembless et al., 2002; Ha and Nemeth, 1995) did not find significant changes in work zone crash severity as compared to non-work zone crashes. The work zone crashes were even found less severe in a few other studies (Garber and Woo, 1990; Rouphail et al., 1988; Wang et al., 1996). Garber and Zhao (2002) observed that nighttime work zone crashes were found to be much more severe in most cases as compared to daytime crashes.

### *2.1.4 Causal factors*

Most previous studies indicated the human errors, such as inattentive driving, and misjudging, as the most common causes for work zone crashes (Bai and Li, 2006; Chembless et al., 2002; Daniel et al., 2000; Mohan and Gautam, 2002; Pigman and

Agent, 1990). Hill (2003) observed a significant difference on types of driver errors as a function of time of day. Other studies indicate that speeding (Garber and Zhao 2002) and inefficient traffic control (Ha and Nemeth 1995) are major factors causing work zone crashes. In addition, according to Pigman and Agent (1990), traffic congestion, construction equipment and materials are factors that increase travel delays together with the crash frequency.

The study conducted by Dissanayake and Akepati (2009) showed that, out of the 720 work zone fatalities occurred in U.S. in 2008, speeding was a factor in 31% of cases and alcohol in 20%. Furthermore they found the lack of seatbelt use as a contributing factor in 53% of fatalities.

## 2.2 The effects of work zone presence in crash frequency

Crash frequency is usually used as a safety evaluation measure for work zones and is expressed as the total number of crashes in a given time period.

Over the past 40 years, many researchers have examined the impact of work zones on roadway safety in terms of crash frequency. Most part of crash investigations agree that highway work zones significantly increase crashes rates as compared to the pre-work zone conditions (Garber and Zhao 2002; Khattak and Council, 2002; La Torre et al., 2014; Pal and Sinha, 1996; Saleh et al., 2013; Srinivasan et al., 2011; Wang et al., 1996).

Several studies show that work zones have an increasing effect on crash frequencies when compared to “pre-work zone” conditions (Graham et al., 1977; Hall and Lorenz, 1989; Juergens, 1972; Khattak and Council, 2002; Liste et al., 1976; Nemeth and Migletz, 1978; Ozturk et al., 2014; Rouphail et al., 1988; Pal and Sinha, 1996; Srinivasan et al., 2011).

Ozturk et al. (2014) used data from 60 work zone sites in New Jersey between 2001 and 2011 in order to examine the work zone and non-work zone crash risk. They found that the average number of crashes and crash rates increased by 18.8% and 24.4% respectively during work zone activities and that rear-end crash frequency was 8.6% higher compared to non-work zone conditions.

The study conducted by the University of Florence as a contribution to the ASAP project (Saleh et al., 2013) showed that the overall expected crash frequency, during the time when a work zone is installed on a motorway segment, is about 32% greater than the crash frequency on the same motorway segments in the “pre-work zone” period.

Srinivasan et al. (2011) investigated 64 freeway construction projects in four different states and reported an increase in crash rate of about 65% when work activity is occurring and travel lanes are temporarily closed.

The study conducted by Khattak and Council (2002) showed that the total crash rate in the during-work zone period was 21.5% higher than the pre-work zone period and that the increases in PDO and injury crash rate were respectively equal to 23.8% and 17.3%y. Also Pal and Sinha (1996) found that crash rates in work zones in Indiana were significantly higher than those in “non-work zones” conditions. Furthermore, the outcomes of their research indicated that the average severe crash rate in work zones with a crossover between the two carriageways was generally higher than for partial lane closures.

### *2.2.1 Crash Modification Factors for work zone studies*

Researchers applied different methodologies to address problems related to work zone safety analysis.

Crash modification factors (CMFs) and crash modification functions (CMFcns) are two related measures typically used for evaluating the safety of work zones. CMF is a multiplicative factor used to compute the expected number of crashes after implementing changes at a specific site, such as implementing a work zone. CMFcn is a continuous function that varies the crash modification factor across a range of variables or combinations of variables. The Highway Safety Manual (HSM) in Section 16.4.2 provides CMFs for all crash severities as a function of the duration and the length of work zones (AASHTO, 2010). Specifically, the HSM synthesized a previous research (Khattak and Council, 2002) in order to provide quantitative evaluation of work zone safety. The CMF related to the work zone duration is given by

$$CMF_{d,all} = 1 + (\%\ increase\ in\ duration \cdot 1.11)/100 \tag{2.1}$$

The CMF related to the work zone length is given by

$$CMF_{l,all} = 1 + (\%\ increase\ in\ length \cdot 0.67)/100 \tag{2.2}$$

The base condition of the CMFs (CMF=1) is a work zone duration of 16 days and a work zone length of 0.84 km. The study conducted by Khattak and Council considered by the HSM investigated crash rates in the “pre-work zone” and “during-work zone” periods by using a dataset of California freeway work zones that included crash data and work zone data during a 2-year period (1992-1993). They considered work zone durations from 16 to 714 days, work zone lengths from 0.80 to 19.63 km and freeway Annual Average Daily Traffic (AADT) from 4,000 to 237,000 veh/days.

In order to account for the combined effect of work zone length and durations the two CMFs should be multiplied as follows:

$$CMF_{total} = CMF_{l,all} \cdot CMF_{d,all}$$

This method takes into account only the combined effect of length and duration, but not the other possible variables involved.

The studies of CMFs and CMFcns can be divided into the two categories of experimental and observational (Carter et al., 2012). Experimental studies are implemented in a laboratory context where researchers can intentionally design an experiment in a desired way in order to answer a certain question. However, in observational studies, the parameters of the study cannot be entirely controlled by the researchers. A common type of observational study is the before-after study, in which the safety performance of a site before a treatment is compared with the performance of the same site after the application of a treatment.

The most common approaches used to perform a before-and-after study are the naïve before-and-after study, the before-and-after with comparison group and before-and-after study with Empirical Bayes approach.

### *2.2.2 Naïve before-and-after study*

In the naïve before-and-after method, crash counts in the before period are considered equivalent to the expected crash counts if the safety treatment had not been implemented. The change in crash counts between the before and the after conditions is considered the treatment effect. This approach ignores the fact that treatment (for example the work zone placement) is not the only factor that might cause changes between the before and the after period and is unable to separate the treatment effect from the other effects such as exposure, trend and regression-to-the-mean effects.

**Exposure effect**

The exposure effect is caused by change in traffic volume and patterns on a facility. Traffic volume and accident frequencies have a direct relationship and crash frequency usually increases as traffic volume increases and vice versa. This effect could be significant if the treatment applied to the site significantly changes the capacity of the site. For example, the installation of a work zone typically causes significant reductions in roadway capacity, especially in the case of construction and maintenance activities.

**Trend effect**

The trend effect is due to causal factors that are not recognized, measured and understood. For example, driver habits, crash reporting practices, enforcement level, weather conditions, vehicle technologies or safety systems can be changed from the before to the after period and can therefore induce significant changes in the overall crash frequency.

**Regression-to-the-mean effect**

Regression-to-the-mean (RTM) is the natural tendency for an abnormally high number of accidents to return to values closer to the long term mean; conversely abnormally low numbers of accidents tend to be succeeded by higher numbers. Regression-to-the-mean occurs as a result of random fluctuation in the recorded number of accidents around the long-term expected number of accidents (Elvik & Vaa, 2004).

In practice, there is a tendency to select sites with high accident frequency or rates for safety treatments. RTM effects arise when sites with high short term crash counts are selected for treatment and experience a subsequent reduction even if no treatments are implemented. The effect of the treatment installed at these locations would be overestimated if the RTM bias is not properly addressed in the analysis.

A change in crash frequency from the before to the after periods is therefore expected even if no safety treatments are applied to a site. Consequently, specific evaluation techniques are required to extract the treatment effect from the total change in safety performance in order to assess if the operational treatment has resulted in a safety improvement.

### *2.2.3 Before-and-after study with comparison group*

A before-after with comparison group study uses an untreated comparison group of sites similar to the treated ones to account for changes in crashes unrelated to the treatment such as time and traffic volume trends (Gross et al., 2010). The comparison group is used to compute the ratio of observed crash frequency in the after period to that in the before period. The observed crash frequency in the before period at a treatment site group is then multiplied by this comparison ratio to provide an estimate of expected crashes that would have occurred at the treatment group sites without treatment applied. This is then compared to the observed crashes in the after period at the treatment site group to estimate the safety effects of the treatment.

Ideally, the comparison group sites should be similar to the treatment sites in terms of geometric and operational characteristics and should not have undergone any geometric change or traffic control improvement during the before and after periods. The larger the comparison group, the better the assessment. However the comparison group could be too small if most or all sites are treated or at least affected by the treatment. Furthermore, this technique cannot determine the treatment effectiveness if accident count in either the before or the after period in the comparison group is equal to zero.

This method is not able to account for RTM unless treatment and comparison sites are also matched on the basis of the observed crash frequency in the before period. Specifically, a control site would need to be matched to each treated site based on the annual crashes in the before period. However, there are important practical difficulties in achieving an ideal comparison group to account for the RTM (i.e., matching on the basis of crash occurrence).

### *2.2.4 Before-and-after study with the Empirical Bayes approach*

The Empirical Bayes (EB) approach (Hauer, 1997) can be applied to properly account for the effect of RTM in addition to traffic volume changes and time trends in crash occurrence.

In accounting for regression-to-the-mean, the number of crashes expected in the before period without the treatment ($N_{expected,T,B}$) is a weighted average of observed crashes ($N_{observed,T,B}$) and predicted crashes ($N_{predicted,T,B}$) in the before period at the treated sites.

Crash prediction models (CPMs) are used to estimate the number of crashes predicted at treated sites ($N_{predicted,T,B}$). CPMs are regression models that explain the relationship between crash frequency and some explanatory variables such as traffic or physical characteristics of sites.

The empirical Bayes estimate of the expected number of crashes without treatment ($N_{expected,T,B}$) is computed as follows:

$$N_{expected,T,B} = N_{predicted,T,B} \cdot w + N_{observed,T,B} \cdot (1-w) \tag{2.3}$$

Figure 2.2 shows how the CPM estimate is weighted with the observed crash count to estimate $N_{expected,T,B}$. The regression-to-the-mean effect is the difference between $N_{observed,T,B}$ and $N_{expected,T,B}$.

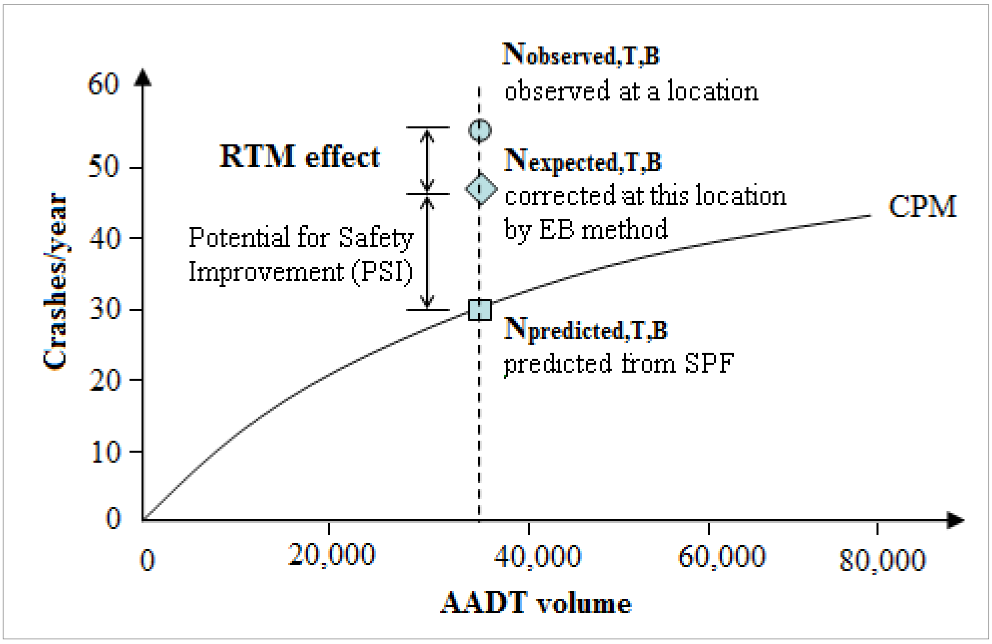



Figure 2.2: Illustration of RTM effect and Empirical Bayes estimate

The prediction model weight (w) is derived by using the over-dispersion parameter (k) given for the specific model used and also depends on the number of years of crash data in the period before treatment. There is an inverse relationship between the prediction model weight and the over-dispersion parameter.

The over-dispersion parameter provides an estimation of the dispersion of the data. Over-dispersion occurs when the variance of observed data is higher than the variance of predicted data. Conversely, under-dispersion means that less variation in the observed data occurs as compared to the predicted. Specifically, in case of low values of the over-dispersion, more weight is placed on the crashes predicted from the model and less weight on the observed crash frequency. The weighted adjustment factor $w$ is computed as follows:

$$w = \frac{1}{(1 + k \cdot \sum N_{predicted,T,B})} \tag{2.4}$$

### *2.2.5 Summary*

Table 2.2 shows a summary of the abilities of each of the three before-and-after methods to address the main confounding factors attributed to a change in safety performance.

Table 2.2: Summary of Before-and-After methods

| **Methodology** | **Ability to account for:** | | | |
|---|---|---|---|---|
| | **Treatment effect** | **Exposure effect** | **Trend effect** | **RTM effect** |
| Before-and-After with Empirical Bayes | Yes | Yes | Yes | Yes |
| Before-and-After with Comparison Group | Yes | Yes | Yes | No |
| Naïve Before-and-After Study | Yes | Potential | No | No |

## 2.3 The role of speed in work zone crashes

Speed is one of the basic risk factors in traffic. High speed reduces the time available for stopping and crash avoidance and increases crash risk. Higher driving speeds lead to higher collision speeds and thus to more serious consequences, in terms of personal injuries and material damage.

According to Wramborg (2005), the chances of survival for pedestrians, cyclists or unprotected workers hit by a vehicle decreases rapidly at speeds greater than 30km/h. As shown in Figure 2.3, unprotected workers have a 90% chance of survival when struck by a car travelling at 30 km/h, but less than a 50% chance of surviving a 50 km/h.

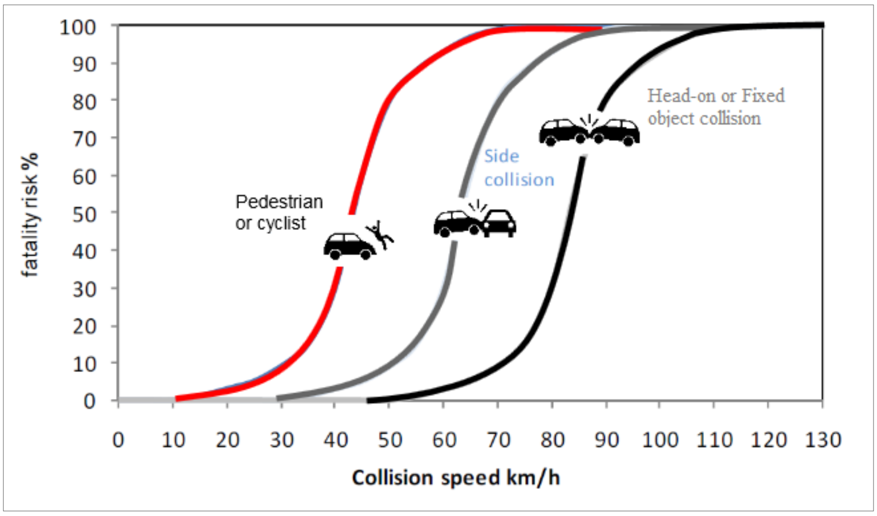



Figure 2.3: Probability of fatal injury as a function of collision speed (Wramborg, 2005)

In the case of side collisions the death risk for car and work vehicle occupants increases rapidly above 50 km/h, whereas in the case of head-on crashes the critical speed is 70 km/h.

Several studies found speeding as a major factor in traffic accidents and fatalities within work zones (Bryden et al., 2000; Dissanayake and Akepati, 2009; Garber and Zhao, 2002; Li and Bai, 2007; Li and Bai, 2009). A report by the Kansas State University (Dissanayake and Akepati, 2009) shows that speeding was a factor in 225 out of the 720 work zone fatalities occurred in the U.S in 2008. In a study of work zone crashes in Kansas (Li and Bai, 2009), speeding was a factor in 15% of the fatal crashes and 20% of the crashes causing injuries. In 2012, the Fatality Analysis Reporting System (FARS) reported that out of the 30,800 fatal crashes recorded in the U.S., 547

(1.7%) occurred in work zones and among the work zone fatalities, speeding was indicated as a contributing factor in 192 (35.1%).

The relationship between speed level and crash severity is an important factor for work zone safety as high speeds imply an increased injury risk for both road users and road workers.

Shaw et al. (2015) identified a relationship between operating speed and safety as a function of the traffic conditions. Specifically they distinguish between work zones operating under stable traffic flow conditions and those under unstable flow ("stop-and-go" traffic). Reducing speeds can be expected to improve safety when the traffic flow is stable. When conditions become unstable average speeds may decline sharply, but crash rates may increase as a result of abrupt fluctuations in the actual speeds. Unstable traffic flow occurs frequently in work zones when the traffic demand exceeds the capacity. Transitions from freely flowing traffic upstream to stop-and-go conditions in the work zone can be critical, particularly if the speed change is abrupt, inconsistent with driver expectations, or occurs under conditions that limit visibility.

While excessive speed (exceeding the temporary speed limit) is usually the main issue in work zones that are operating under stable traffic flow conditions, inappropriate speed (driving too fast for prevailing conditions) contributes to crashes in work zones with stop-and-go traffic. In work zones on multilane roadways, highly aggressive drivers may attempt to exceed the prevailing speed by making frequent, abrupt lane changes into the fastest moving lanes. If an aggressive driver misjudges the headway or the traffic speed, the risk of rear-end collisions increases.

In work zone-related crashes, the analysis of the speed variance, in addition to the analysis of the mean speed, can therefore provide more relevant information. A large speed variance may lead to higher accident rates at work zones: the relationship between travel speed and accident rates indicates that accident rate increases as speed variance increases (Garber and Gadiraju, 1998; Migletz et al., 1998; Salem et al., 2006).

The study conducted by Migletz et al. (1998) highlighted an important relationship between speed limit reduction, speed variance and fatal + injury crash frequency in work zones. Their findings showed that compliance with work zone speed limits was generally higher where the speed limit was not reduced and decreased where the speed limit was reduced by more than 16 km/h. Specifically they recorded an average decrease of 8.2 km/h in actual speeds in the cases without speed limit reduction and a mean reduction of only 20.4 km/h when the work zone limit was set 40 km/h below the ordinary limit (Figure 2.4).

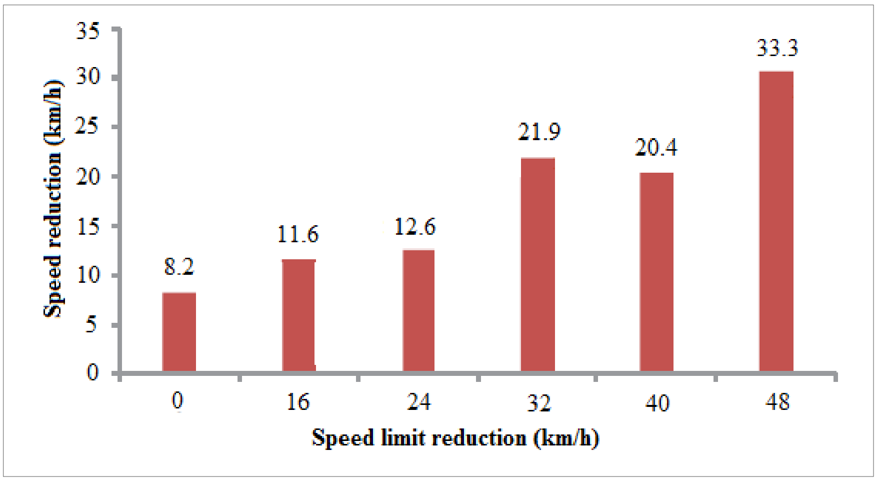



Figure 2.4: Mean speed reductions from upstream to work zone locations (Migletz et al. 1998)

In freeways work zones without a speed limit reduction, the percentage of vehicles exceeding the speed limit was in general lower inside the work area than upstream on the average by 21.7% (Migletz et al., 1998) as shown in Figure 2.5.

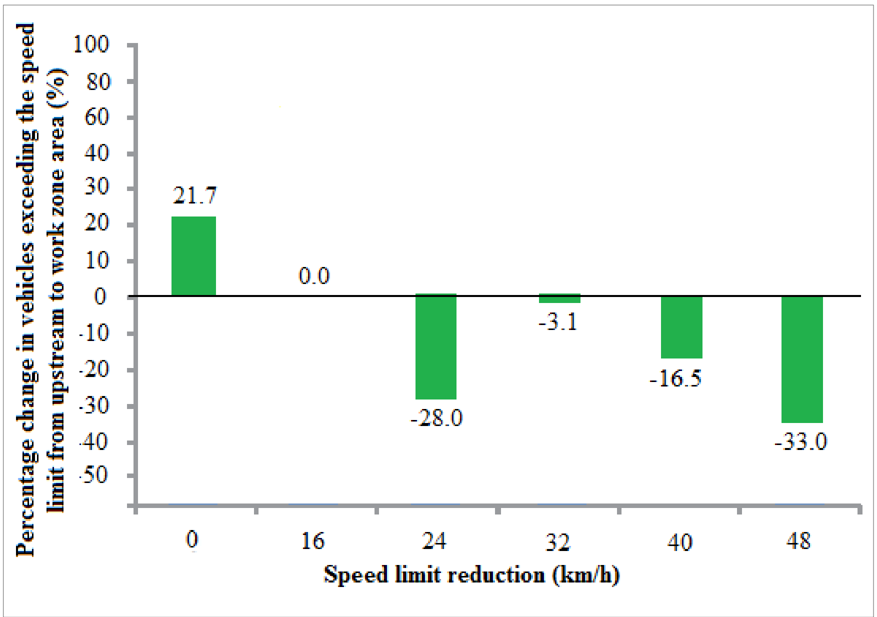



Figure 2.5: Change in percentage of vehicles exceeding the speed limit from upstream to work zone (Migletz et al., 1998)

Furthermore they noticed that the percentage increase in speed variance, from upstream to work location, appears to have a minimum for a speed limit reduction of 16 km/h. For work zones without a speed limit reduction, the speed variance in the work zone was 62% higher than the upstream speed variance (Figure 2.6).

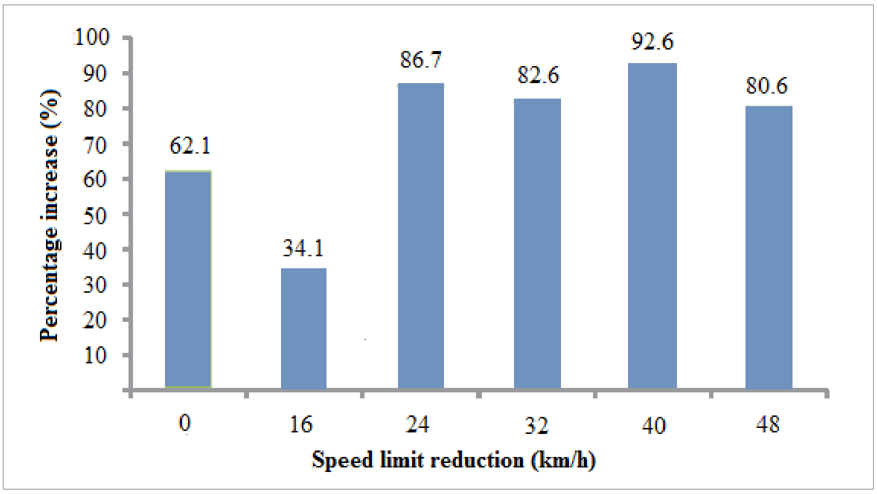



Figure 2.6: Percentage increase in speed variance from upstream to work zone areas (Migletz et al., 1998)

According to Figure 2.7, the minimum percentage of fatal-plus-injury accident rates during the construction period occurs for a speed limit reduction around 16 km/h and the next smallest percentage increase in the fatal-plus-injury accident rate occur in work zones without speed limit reductions.

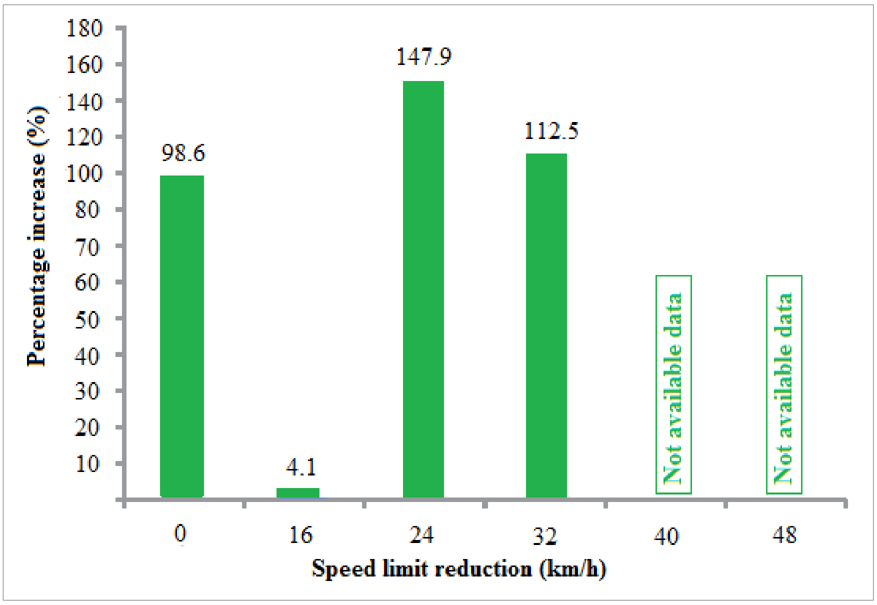



Figure 2.7: Percentage increase in fatal + injury crashes from “pre-work zone” period to “during-work zone” period (Migletz et al., 1998)

The results presented in Figure 2.7 are noteworthy because they show a similar trend to that showed by the results of the speed variance analysis (Figure 2.6). This means that the safest traffic flow conditions occur when all vehicles are travelling at approximately the same speed, thus when the speed variance is small. Specifically the safest work zones are those with the smallest increase in the upstream-to-work-zone speed variance.

## 2.4 Speed management strategies through work zones[1]

Several speed control techniques are currently used worldwide to improve speed limit compliance. This section provides a detailed review of safety methods used to improve compliance with speed limits in work zones. These methods can be informational measures (such as signs and flaggers), physical systems (such as rumble strips, chicanes, channelizing devices), enforcement (such as police presence and automated control) and perceptual countermeasures.

### *2.4.1 Informational measures*

One method to address appropriate speed limits is to provide motorists with information related to work zones, speed limits, penalties for traffic law violation, real-time speed feedback of individual motorists, and hazard warnings. The measures commonly adopted in this area are:

- Regulatory speed limit signs;
- Speed monitoring displays;
- Variable message signs;
- Other solutions.

**Regulatory speed limit signs**

Posted speed limit reductions in work zones are the primary means of advising drivers that a reduced speed is either in effect or is advisable and also provide the legal basis for enforcement. Alternatively, advisory speed limit signs indicate a recommended safe speed through temporary work zones. Advisory speed limit signs are generally used as supplementary warnings of an approaching hazard.

The solution to work zone speed management is not simply posting low speed limits. It has generally been found that control of traffic speeds by imposing

[1] This chapter contains a summary of methods used to manage and control vehicle speed in road work zones. The description of such methods was directly extracted from the deliverable 2.1 of the ASAP Project - “State of the Art on Speed Management Methods” (Nocentini et al., 2013), in which over 270 technical documents were collected and reviewed.

unwarranted regulatory speed limits has not been very effective. It is the situation they see and not the reduced speed limit that cause drivers to reduce their speeds (FHWA, 2009). Two factors should be taken into consideration on drivers' speed choice:

- *voluntary reduction*: is lower than 16 km/h, often closer to 8 km/h (unless the presence of enforcement) and is due to the presence of devices or to the assertion from authorities to reduce speeds.
- *involuntary reduction*: depends on "what the driver sees" in their field of view. The driver slows down only if there is the perceived need to do so. This is based on conditions in the work zone or the perception of enforcement activities. Typically the drivers slow down when large equipment and work crews are located close to the travelled way, when a roadway restriction occurs (temporary crossovers or narrowed lanes) or temporary traffic barriers are near to the edge of the lane.

An effective measure to improve "speed behaviour" (compliance to speed limits) is to increase the credibility of speed limits. It is assumed that when speed limits are more credible, the speed limit is in line with the expectations and drivers are better inclined to comply with them (SWOV, 2012). Advisory speed plaques and supplement warning signs should therefore be considered before deciding to lower the speed limit. If drivers do not perceive a hazard, they will not reduce their speeds. Similarly, if the advisory speed seems excessively low, drivers will not slow down to that speed. Of greater concern, they will lose confidence in other signs where the speed may be realistically posted, thus failing to adhere to the advisory speed. When the speed limits are credible a positive effect is expected on average driving speeds or speeding and on homogeneity of the traffic flow. Imposing a work zone speed limit that drivers perceive to be unreasonably low has been shown to increase speed variation: conservative drivers tend to observe the work zone limit, whereas aggressive drivers may attempt to speeds closer to the ordinary limit (without road work).

Most of the national standards state the work zone speed limit reductions should be avoided, when possible, where all work activities are located on shoulder or roadside areas and when workers are not present. Several research studies show that it is difficult to achieve an average speed reduction of more than 15 km/h. Posted speed limits in work zones should not be more than 20 km/h below the normal posted speed limit for that road section, except where required by restricted geometrics or other work zone features that cannot be modified.

Experience has shown that the use of signs to reduce the speed of traffic through work zones has varying degrees of effectiveness. A study conducted by Colorado Department of Transportation (DOT) (Outcalt, 2009) has shown that long-term work zones with significant speed reductions and where workers and equipment are far from traffic tend to make drivers doubt the credibility of the posted speed limit. This study showed that drivers reduce speeds in work zones, particularly when workers are present, independently of whether or not speed limit reductions are posted.

A 16 km/h maximum speed reduction is still warranted for lane closures and temporary diversions. Speed limit reductions should be discouraged on roadways with existing speed limits less than 104 km/h for all conditions except in case of lane closures when workers are in a closed lane which is not physically protected by a barrier and only a single travel lane remains open. Short term work zone speed limits are posted only when work activity is present. When the work activity is not present, the short term work zone speed limit signs should be removed or covered.

Variable Speed Limits (VSLs) generally have more influence on speed reduction than traditional static signs (Figure 2.8).


Figure 2.8: Variable Speed Limits (La Torre et al., 2014)

VSL can be used to alter the speed limit considering the on-going conditions in the work zone (e.g., as workers presence or weather conditions) posting a reasonable speed limit, based on real-time traffic flow, roadway and speed conditions.

In U.S. the Intelligent Transportation Systems program has given new impetus to implementation of variable speed limit systems. The Federal Highway Administration (FHWA) supports additional development of variable speed limits in work zones, and has awarded funding for field tests in Michigan, Maryland, and Virginia. These field tests do not use a fixed posted speed, but measure real-time traffic, then compute and post a speed limit reflecting the safe speed at which drivers should be travelling. As a general rule, if a variable speed limit sign lowers the speed limit because workers or other hazardous conditions are present, the hazards should be evident to drivers. Enforcement agencies need to be informed of changes in the speed limit in order to effectively provide speed limit enforcement and to document the speed limit that is in place when the fine is issued.

Based on a test conducted to investigate the use of VSL signs in the State of Utah (Riffkin et al., 2008) the response and long term application of VSL signs is very positive. VSL led to lower average speeds than static speed limits signs through the construction zone and also the variance in speed distribution was reduced. Following the results of this research it was recommended that VSL signs should be placed with the same spacing as the static signs, which may result in the need for additional VSL signs and higher construction costs.

When demand volumes are extremely high, VSLs offer no appreciable benefit over static speed limits. Furthermore, a cost/benefit analysis indicated that VSLs may be mostly appropriate for long-term applications (Fudala and Fontaine, 2010).

**Speed monitoring displays**

Speed monitoring displays (SMD), also known as mobile radar trailers, were developed in the late 1980s (Figure 2.9).


Figure 2.9: Speed Monitoring Displays (La Torre et al., 2014)

SMDs are usually stand-alone systems that can be placed individually, or in a series. The system consists of a self-contained trailer unit equipped with radar to measure the speed of approaching vehicles. The display boards are generally not used to enforce the speed limits. Approaching vehicle speeds are displayed on LED panels along with the posted work zone speed limit, and a message stating “Your Speed”. The systems are typically battery powered to last at least one week. These speed reductions are assumed to occur in two ways: drivers read the display, realize that they are speeding and choose to slow down otherwise motorists with radar detectors will be likely to slow down when their detectors are activated by the radar signals.

Speed display units may be used in any type of work zone, but because of the cost and installation requirements they are mostly appropriate on roadways with higher volumes or speeds. Several studies have been conducted to evaluate the effectiveness of SMDs in work zones.

A Swedish study showed that speed displays on slow moving work vehicles had a major impact on speed, with a mean speed reduction of 24 km/h and an 85 percentile reduced by 47 km/h. The signs are particularly useful on motorways and larger rural roads (Kalman and Sjöholm, 2010)**.**

Recent studies have proven the long-term effectiveness of radar speed monitoring displays. Portable trailer mounted displays are appropriate for temporary speed reduction needs such as work zones. Long-term speed management needs however are better served with a permanently mounted speed monitoring display (Bowie, 2003).

Pesti and McCoy (2001) evaluated the long-term effects of speed monitoring displays. Three display trailers were deployed for five weeks in two work zones on an interstate highway in Nebraska. The mean and the standard deviation of approach speeds and the percentage of vehicles complying with the speed limit were evaluated. Researchers determined that display trailers were effective in lowering speeds, increasing uniformity of speeds, and increasing speed limit compliance.

**Variable message signs**

A commonly used device to increase driver awareness in work zones is a text Variable Message Signs (VMS, Figure 2.10). VMSs can provide drivers with real-time information about conditions, and can be particularly useful at work zones where unexpected traffic or detour situations exist.


Figure 2.10: Variable Message Signs (ASAP, 2013)

The decision to use VMSs is based on a number of factors including availability, reliability of equipment, and installation and maintenance costs. VMS effectiveness on reducing speeds is strictly connected to placing a message on the sign only when there is a specific activity or condition that really requires the message. The signs should be capable of being operated remotely with a list of messages developed prior to the beginning of the construction activity. The number of signs and the distance of the first sign from the construction site would be dependent on the characteristics (speed, traffic volume, roadway geometries) of the specific work zone.

In 1995, a research team examined the effects of the following four messages in Virginia work zones (Garber and Patel, 1995):

- “you are speeding, slow down”;
- ”high speed, slow down”;
- “reduce speed in work zones”;
- “excessive speed, slow down”.

When comparing driver responses to each of these four messages, there was not a significant statistical difference in the ability of the messages to reduce vehicle speeds. The four messages examined produced speed reductions of 8 to 16 km/h. The first message was the most successful on reducing the mean and the 85th percentile of speeds. In addition, the speed variance between drivers decreased. This message successfully singled out drivers, and the words “you are” conveyed the meaning that this message was not a general warning.

Similar results were observed by Clemson Researchers (Sarasua, 2006), that tested four different sequences to be shown on the display of a VMS. The percentage of drivers exceeding the speed limit was reduced for all messages and the message “you are speeding” was proven to be the most effective in reducing speeds in work zone areas.

**Other solutions**

In Finland, Denmark and Sweden “particular messages” were tested such as ‘Take care of my father’ and ‘Here my father is making way for you’ or ‘My father works here’ and ‘Take care of me I work for you’. These kinds of messages seem to have very beneficial effects on reducing speeds (Figure 2.11).


Figure 2.11: Examples of “particular messages” (La Torre et al., 2014)

The results of velocity measurements show that video surveillance has the potential reducing speed past the work zones (Bolling and Nilsson, 2001). Measured rate effects were in the order of 5 km/h when the speed was measured before and after installation of monitoring equipment and associated signage. It is worth noting that the speed of the effect was independent of the rate level, i.e. deceleration was 5 km/h over the speed range. The decrease in speed when the camera used meant that the number of speeding violations approximately was halved. From the questionnaire responses from the speeders one can see that they were positive about the camera supervised work zone sites.

### *2.4.2 Physical devices*

Physical devices are used to influence motorists’ speeds by placing traffic calming devices on the road surface which generate sound, vibration or optical illusion that affects drivers’ perception of speed. The measures commonly adopted in this area are:

- Channelizing devices;

- Portable Rumble Strips;
- Chicanes.

**Channelizing devices**

Channelizing devices such as delineators, traffic cones, drums, channelization curbs, tubular markers and temporary barriers (Figure 2.12) are commonly used to separate the traffic flow from either the work space or opposing lanes of traffic. In major work zones concrete barriers can also be used.

Figure 2.12: Channelizing devices (La Torre et al., 2014)

These devices are also extensively used as a method to reduce lane widths and therefore to encourage drivers to moderate their speeds. In general, narrower lanes leave less lateral distance between vehicles in adjacent lanes or between vehicles and shoulder obstructions, increasing motorists' attention and inducing motorists to reduce speeds. Lane narrowing also presents a relatively inexpensive form of speed control for long-term projects because of the modest maintenance costs. Although inexpensive and relatively easy to implement, narrowing lane widths can reduce roadway capacity. There is also a greater possibility of vehicles striking the cones or other devices, which could increase the number of crashes in these work zones (Trafikverket, 2011).

In North American experience, lanes that are too narrow (less than 3.0 m) may lead to driver discomfort, difficulty in remaining within the lane, and increased collisions, especially for trucks (Harmelink et al, 2005).

Results of extensive literature review conducted by Saleh et al. (2013) as a contribution to the ASAP project, concluded that narrower lane widths increase the risk of lateral crashes and recommended a minimum width equal to 2.75 m for lanes for cars and a minimum width of 3.25m for lanes for trucks. However, too wide lanes (>5m) can lead to uncertain track behaviour and should be avoided.

In Switzerland, Spacek et al. (2005) investigated the effects of various channelization devices in four different work zone crossovers. The results indicated that when travel directions were structurally separated from the work zone activity area by concrete barriers, the crash rate was roughly the same of that recorded in the situation without work zone. Furthermore the findings showed that channelization curbs caused smoother decelerations in approaching the crossover compared to the configurations with vertical delineators.

**Portable Rumble Strips**

Temporary rumble strips are self-adhesive strips that create an audible, visual, and physical alert when driven over (Figure 2.13). These brightly coloured strips are intended to warn drivers of an approaching work zone where they may be required to stop, merge, or simply slow down.

Figure 2.13: Rumble strips (La Torre et al., 2014)

Heaslip et al. (2010) conducted tests on portable plastic rumble strips and reusable temporary rumble strips made out of steel with a rubber bottom placed on a closed roadway surrounding Kansas City. They noticed that plastic rumble strips moved such a large amount when traversed by heavy trucks and concluded that they should be avoided for use at work zones.

The Indiana, Maryland, Utah, and Arkansas DOTs tested rumble strips at several locations and found that the strip cracked easily and moved when trucks passed over it and also noted that some drivers swerved around the strip to avoid it. None of the DOTs that studied the portable rumble strip recommended its use (Trout and Ullman, 1996).

**Chicanes**

A chicane is a traffic management solution characterised by a change in the alignment of the traffic flow on the carriageway. A motorist passing through a chicane is forced to change directions twice in quick succession, typically reducing the speed to do so. A study conducted by Nygårdhs (2007) compared three different types of configurations for chicanes (Figure 2.14).

Figure 2.14: Different types of equipment arrangements (Nygårdhs, 2007)

A comparison with conventional equipment used in Sweden was also conducted. The configuration with a white barrier was considered more clear than the other configurations even though it resulted in higher speeds.

### *2.4.3 Enforcement measures*

Enforcement measures are used to enforce speed limits by automated speed monitoring, speeding detection, imposition of violation fines, and presence of police cars. In general police enforcement is perceived to be one of the most successful work zone speed reduction strategies. A Canadian (Harmelink, 2005) survey regarding the effectiveness of police enforcement, indicated that 55% of respondents rated their effectiveness as high, 30% as moderate, 10% as low, and 5% as not effective. In general, most speed reduction measures are likely to be more effective if they are supported by police enforcement. There are also some speed reduction measures that are unlikely to be effective unless supported by some level of police enforcement. Measures that have proven to be effective in helping to manage speeds in work zones include police presence and enforcement within the work zone and automated speed enforcement.

**Police in work zone**

An observable police enforcement strategy can involve mobile or stationary police cars. In general a police officer stationed at one point significantly increases the speed limit compliance at that location. On the other hand, a circulating police car covers a larger area but is less effective at speed reduction.

The long-term effects of police presence were evaluated in Michigan on Interstate 96 (Sisiopiku and Patel, 1996). This study indicated an average speed reduction of 8.8 km/h for vehicles approaching a parked police car. Upon passing the police car, however, drivers tended to return to original speeds or higher. The study reported no discernible changes in speeds one, two, and three hours following police presence.

A study by Hajbabaie et al. (2009) compared the effects of four speed management techniques on speed on interstate highway work zones. The techniques are a speed feedback trailer, a police car, a speed feedback trailer plus a police car and automated speed photo-radar enforcement (SPE). The results showed that all enforcement treatments significantly reduced the mean speeds and the rate of speeding drivers. Specifically the implementation of the trailer plus police presence reduced the mean speeds more than the other treatments.

The previous results demonstrate that a combination of more than one device could improve speed compliance through the work zone. The location of police vehicles in relation to the work zone also needs to be considered. This could be in advance of the work zone (upstream), within the work zone and beyond the work zone (downstream). Generally, it is beneficial to position police vehicles upstream or at the beginning of the work zone, because of its powerful effect to moderate speeds before entering the work zone (FORMAT, 2004).

**Automated speed enforcement**

Automated speed enforcement devices utilize a radar or laser devices to detect speeds of oncoming traffic (Figure 2.15)


Figure 2.15: Automated speed camera (La Torre et al., 2014)

The device takes a picture of the vehicle's license plate (and of the driver if needed in certain jurisdictions). With this solution officers do not have to pursue, or attempt to pull over, vehicles within the work zone.

Spot enforcement can be labour intensive and costly with long term use and semi-mobile speed cameras (installed for several days) are more and more used, typically in Belgium for safety sensitive road work sites.

In Maryland three different sites were selected to measure the spatial and temporal effect of automated speed enforcement on motorists' speeding behaviour (Franz and Chang, 2011). For data sets that compared the before versus during analysis periods, the enforcement period displayed a general reduction in aggressive motorists (travelling more than 16 km/h over the posted speed limit). At the same time a more stable spatial speeding distribution through the work zone was induced.

Another, relatively new speed enforcement technique is the average speed control (also called 'section control' or 'point-to-point' control) that records the average speed over a road section. The vehicle is identified when entering the enforcement section, and again when leaving it. The average speed can be calculated based on the time interval between these two points. These systems resulted to be very effective in ordinary motorway sections but usually are not applied in work zones as they require long uniform travel sections (typically of approximately 10 km).

### *2.4.4 Perceptual countermeasures*

Speed perception refers to a driver's judgment of how fast he is travelling. While direct speed information is available from the speedometer, drivers still rely heavily on stimuli from the environment to judge how fast they are travelling. Auditory (engine noise) and tactile (vibrations) information can influence speed perception. However, drivers' primary basis for estimating their speed is the visual sensation provided by the roadway geometric features and other information about objects in their immediate environment streaming through their visual field (Campbell et al., 2012).

The speed chosen by the drivers is mostly an unconscious process, depending on the interaction of the human information processing with the optical density of the field of view. The latter is a function of the visual information and can be defined as the number of objects that contrast with the background. A very small number of contrasting objects leads to monotony and both reduced performance and reactivity. To avoid monotony, the driver subconsciously changes his driving behaviour in order to increase information input: he swerves, brakes or, in most cases, increases speed. An optimal level of optical density, stimulating the driver without overloading him, and the reduction of the perceived spatial depth of the field of view lead unconsciously to slow down (PIARC, 2008). The amount of information to be processed influences the quality of driving (Yerkes-Dodson Law, Figure 2.16) and therefore the driver's speed (Yerkes and Dodson, 1908).

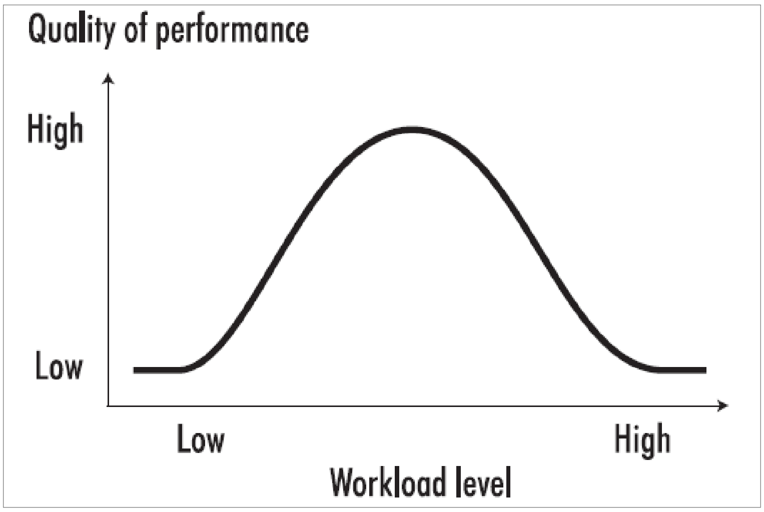



Figure 2.16: Yerkes-Dodson-Law (Yerkes and Dodson, 1908)

Perceptual countermeasures are non-obtrusive means of reducing driving speeds by manipulating the visual environment to induce a perception of higher driving speeds. This approach consists in manipulating the visual environment by means of different traffic calming measures to unconsciously induce motorists to moderate their speed. The driver thus achieves a notion of driving at a "comfortable and safe driving speed" at a lower vehicular speed. The technique typically involves use of pavement markings such as chevrons, traverse lines, herringbone patterns, painted perpendicular to the path of traffic.

Optical speed bars are pavement markings for reducing speeds and decrease the number of accidents in work zones. They are much more common in Europe than Canada or U.S. Gradually decreasing the distance between the strips (Figure 2.17) creates the illusion of speeding and causes drivers to decrease their speed.

Hildebrand et al (2003) evaluated transverse speed bars on one rural highway work zone in a five-week experiment and compared the effectiveness between night and day.

The results showed that the night-time conditions had a greater reduction in speed in comparison to daylight hours.

Figure 2.17: Example of optical bars (la Torre et al., 2014)

Many studies have been conducted with driving simulators to evaluate the effectiveness of this treatment in terms of speed reduction even though they do not specifically refer to work zones.

Godley et al. (1999) conducted several experiments with a driving simulator to evaluate different treatments such as transverse lines and peripheral transverse lines (Figure 2.18). The peripheral bars were the best treatment in terms of cost/benefit ratio.

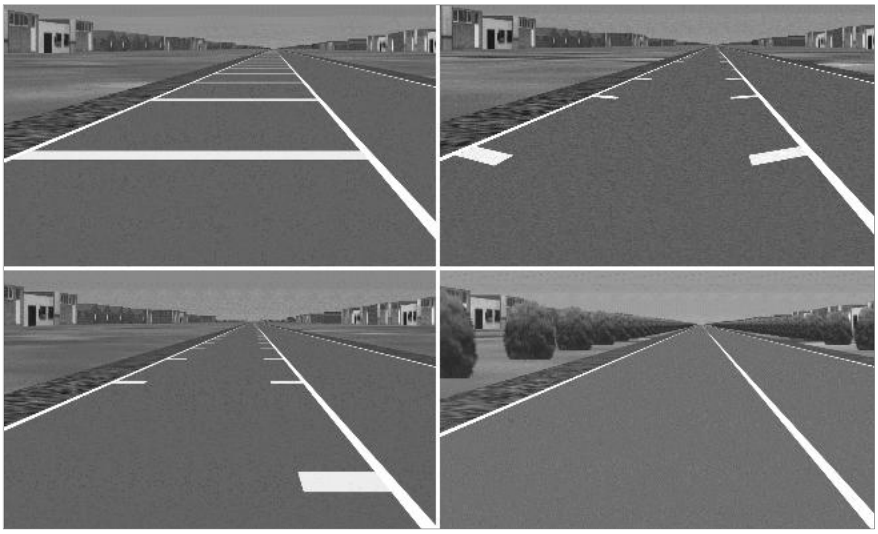


Figure 2.18: Transverse lines (left top); peripheral transverse lines (right top and left bottom); and edge of the road with trees (right bottom) (Godley et al., 1999)

Manser and Hancock (2007) tested different visual patterns applied to transportation tunnel walls in a driving simulator (Figure 2.19). Thirty-two participants experienced three visual patterns consisting of black-and-white vertical segments that decreased, increased or remained constant in width throughout the length of the tunnel. Participants also drove a baseline control condition in which no visual pattern was present.

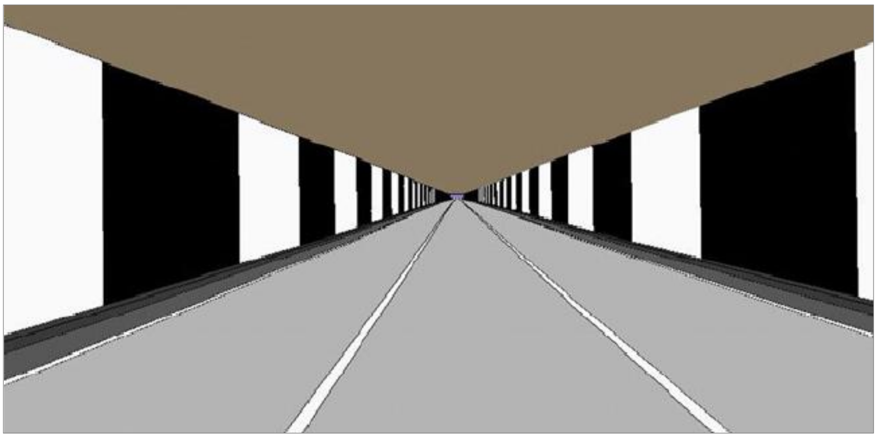


Figure 2.19: Example of experimental condition (Manser and Hancock, 2007)

When compared to the baseline condition, results indicated that drivers gradually decreased speed when exposed to the decreasing width visual pattern and increased speed with the increasing width visual pattern.

### *2.4.5 Summary*

Several methods have been used for controlling work zone speeds. All of them have shortcomings in terms of effects. Some of the most promising are those related to speed monitoring and variable message signs where the driver is provided real time information on their speed or on the traffic situation ahead. Static traffic signs appear to provide some speed management effects but there were no consistent results from the different documents reviewed. Police enforcement had some of the largest effects but only when the police presence was connected to active enforcement activities. The main conclusion from the speed management review was that work zone speeds tended to have more uniform speed distributions, but only small reductions in average speeds without a dynamic system (i.e. variable messages and police enforcement).

## 2.5 Work zone studies with driving simulators

The evaluation of work zone safety measures by means of field tests is costly, difficult to modify, subject to environmental changes and can pose risks for safety of both test participants and researchers. Driving simulators are an effective alternative research tool and allow to evaluate a wide range of interventions that cannot be implemented on site due to legislation restrictions and entailing reduced implementation costs and safer testing conditions.

An extensive literature review carried out by Bella (2009a) showed that driving simulation provides the driver with enough visual information to allow him to correctly perceive speed and distance. In particular, several experimental studies comparing on-road and simulation performance through work zones have revealed a validity of medium-high fidelity driving simulators (Bella, 2004, 2006; Bham et al., 2014, Mc Avoy et al., 2007).

A large amount of researches aimed at evaluating the driving behaviour in approach and within work zones have been carried out with driving simulators in the last decade (Bella, 2009b; Gustafsson et. al, 2014; Mc Avoy et al., 2011; Nelson et al., 2011; Reyes, 2010; Sommers and Mc Avoy, 2013; Ullman et al., 2005, 2007). Most of these studies were aimed at evaluating the effect of different speed management systems on driving performance and focused on the analysis of mean speeds and decelerations.

In 2009, Bella conducted a study to evaluate the driver behaviour close to crossover work zones (Bella, 2009b). Driving simulations were carried out on four different work zone configurations and focused on the analysis of mean speeds and mean decelerations in response to different schemes of signalling and different work zone geometry.

The results indicated that drivers are not affected by the imposed speed limits and travel at higher speeds than that indicated on the traffic sign. The recorded mean speeds were below the limits only within the crossover area.

The study conducted by Reyes (2010) evaluated the effect of work zone barrier type, presence of a lateral buffer, and work zone activity level on measures of speed and lane position of twenty-four subjects in a driving simulator (Figure 2.20).

Figure 2.20: Driver's view of a work zone with lateral buffer (Reyes, 2010)

The subjects drove faster and with less variability in work zones with concrete barriers. Speed was reduced and more variable in work areas with a high level of activity than in areas with a low level of activity. On the whole, the presence of a lateral buffer reduced speed variability in the activity areas but this result was not confirmed across all the configurations.

A more recent study evaluated the potential use of audio warnings at road work sites was carried out in a VTI driver simulator (Gustafsson et. al., 2014). Twenty-two car drivers drove a 25 km-long motorway section with two work zone installed on the outside shoulder (Figure 2.21).

Figure 2.21: Closure of the outside shoulder (Gustafsson et. al., 2014)

Half of the subjects were given an audio warning before the first work zone and the other half prior to the second roadwork. The audio warning consisted of a sound similar to that of a GPS warning signal, followed by a female voice saying "Warning! Road work within 500 meters. Adjust the speed!". The average decrease in speeds was about 9-17 km/h greater with audio warning than without. Furthermore, there was still a small effect (0.5-3.3 km/h) left of the audio alert after the road works.

# Chapter 3.

# Accident analysis in stationary work zones

## 3.1 Introduction

Using a set of motorway work zone data, this analysis provides information on the change in expected crash frequencies associated with the installation of work zones. The aim of this study was to evaluate the effect of different work zones' layout configurations on fatal + injury crashes.

The safety performance of motorway segments before the introduction of a work zone and during the work zone period was evaluated, in order to investigate the work zone impact on the number of expected crashes. The study required information on the work zone layouts, start and end dates, location of work zones, length, crashes during the pre-work zone and work zone periods, and other information such as the annual average daily traffic (AADT) in each segment. Such information was provided by Autostrade per l'Italia S.p.A (ASPI), the largest concessionaire for the construction and management of motorways in Italy. This chapter provides the description of the data used, the statistical methodology, and the results of the analysis. More detailed descriptions of the work zone layouts are included in Appendix A.

## 3.2 Data

### *3.2.1 Analyzed Network*

The first research task in the analysis focused on gathering available data on work zone crashes in Italy. The study used data from the Italian motorway network managed by ASPI to build a comprehensive dataset. The company manages about 3,000 km of motorways distributed along the whole country (Figure 3.1).


Figure 3.1: Motorway network managed by ASPI (web)

### *3.2.2 Databases*

The data sources are crashes, traffic work zones and road inventory data files, provided by ASPI. More specifically four different databases were collected:

- crash database;
- work zone database;
- motorway segment database;
- traffic database.

**Crash database**

The crash database contains details of about 105,000 crashes occurred on the motorway segments from January 1, 2007 to December 31, 2012.

For each crash, several details are provided such as date, hour, localization on the motorway segment, pavement and weather conditions (Figure 3.2). Furthermore it is possible to understand whether or not each accident occurred in the proximity of a work zone. This information is collected by ASPI analyzing police reports on crashes.

| Accident_ID | Motorway | Segment | Location (km) | Date (hour) | Type of crash | Weather conditions | Pavement conditions | Speed limit | in the proximity of a work zone? | Duration (hour) |
|---|---|---|---|---|---|---|---|---|---|---|
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |

Figure 3.2: Crash Database

The database does not specify the exact accident location within the work zone. To confirm if there were anomalies in the database concerning a work zone presence, a consistency check has been conducted in collaboration with the road operator. Crash data and work zone data were compared in order to understand if the accidents actually occurred within a work zone. The accidents that occurred within the work zones resulted in 21 fatalities and 1,252 injuries (in 762 crashes) during a 6-year period from 2007 to 2012.

### Work Zone database

The work zone database provides information about the road works installed in the ASPI network in the same 6-year period from 2007 to 2012 (Figure 3.3).

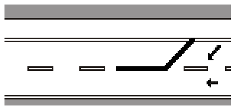
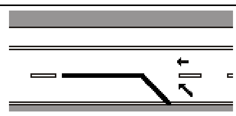
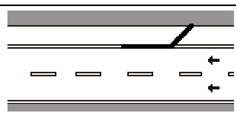
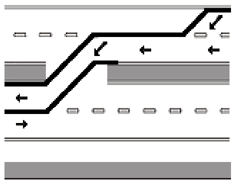
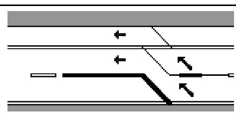
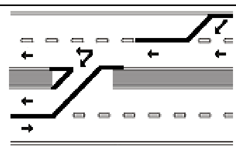
| WorkZone_ID | Motorway | Segment | WZ_begin (km) | WZ_end (km) | Direction | Length (km) | Signalling | Starting date | Ending date | Duration (hour) |
|---|---|---|---|---|---|---|---|---|---|---|
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |

Figure 3.3: Work Zone Database

More than 30,000 stationary work zones were installed on the motorway network from January 1, 2007 through December 31, 2012. The work zones in place for at least twelve hours have been considered as stationary work zones, according to the definition provided by the Italian ministerial Decree 10 July 2002 (Ministero delle Infrastrutture e dei Trasporti, 2002). For each work zone, details on the exact position on the motorway, starting and ending date, the signalling and further details on the layout configuration are provided. However each record of the database does not necessarily correspond to a single work zone but to a single work activity. Very often different work activities can be performed in a single work zone and it was therefore necessary to group the records referred to a single work zone in order to define the exact number.

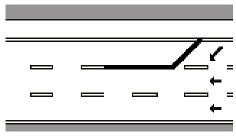
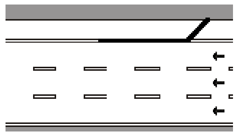
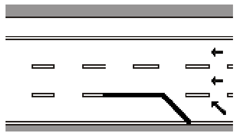
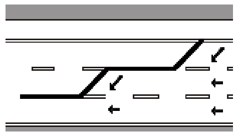
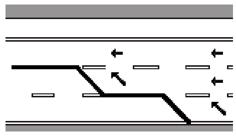
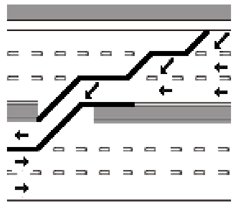
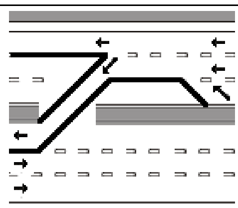
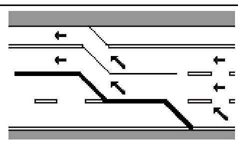
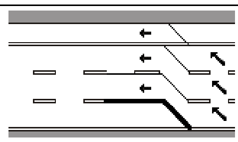
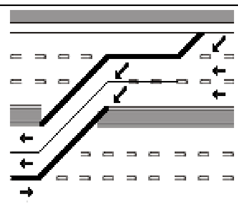
Each work zone is associated to one of the stationary layout configurations (Table 3.1 and Table 3.2), defined according to the Italian Ministerial Decree 10 July 2002 (Ministero delle Infrastrutture e dei Trasporti, 2002). The work zone layouts are illustrated in Appendix A.

Table 3.1: Work zones' configurations for four-lane median divided motorways (two-lane carriageway)

| Stationary work zones | Description | Simplified sketch |
|---|---|---|
| Slow2 | Closure of the slow lane with traffic diverted to the overtaking lane. |  |
| Fast2 | Closure of the overtaking lane with traffic diverted to the slow lane. |  |
| Emergency2 | Closure of the emergency lane (outside paved shoulder). |  |
| Cross2(0+1) | Closure of the slow lane with traffic diverted to the overtaking lane; closure of the overtaking lane and total diversion of traffic to the opposite carriageway through a single-lane crossover. |  |
| Fast2(2) | Closure of the overtaking lane with traffic diverted to the slow and to the emergency lanes. |  |
| Cross2(1+1) | Closure of the slow lane with traffic diverted to the overtaking lane; partial diversion of traffic to the opposite carriageway through a single-lane crossover |  |

**Note: The layouts are sorted in descending order from most frequent (numerous) to least frequent**.

Table 3.2: Work zones' configurations for six-lane median divided motorways (three-lane carriageway)

| Stationary work zones | Description | Simplified sketch |
|---|---|---|
| Slow3 | Closure of the slow lane with traffic diverted to the middle lane. |  |
| Emergency3 | Closure of the emergency lane (outside paved shoulder). |  |
| Fast3 | Closure of the overtaking lane with traffic diverted to the middle lane. |  |
| Slow&Middle3 | Closure of the slow lane with traffic diverted to the middle lane; closure of the middle lane with traffic diverted to the overtaking lane. |  |
| Middle&Fast3 | Closure of the overtaking lane with traffic diverted to the middle lane; closure of the middle lane with traffic diverted to the slow lane. |  |
| Cross3(0+1) | Closure of the slow lane with traffic diverted to the middle lane; closure of the middle lane with traffic diverted to the overtaking lane; closure of the overtaking lane and total diversion of traffic to the opposite carriageway through a single-lane crossover. |  |
| Cross3(1+1) | Closure of the overtaking lane with traffic diverted to the middle lane; closure of the middle lane and partial diversion of traffic to the slow lane and to the opposite carriageway through a single-lane crossover |  |
| Middle&Fast3(2) | Closure of the overtaking lane with traffic diverted to the middle lane; closure of the middle lane with traffic diverted to the slow lane and to the emergency lane. |  |
| Fast3(3) | Closure of the overtaking lane with traffic diverted to the middle lane, to the slow lane and to the emergency lane. |  |
| Cross3(0+2) | Closure of the slow lane with traffic diverted to the middle lane; closure of the carriageway and total diversion of traffic to the opposite side through a dual-lane crossover. |  |

**Note: The layouts are sorted in descending order from most frequent (numerous) to least frequent**

**Motorway segment database**

The motorway segment database contains details on the roadway characteristics of about 2,100 km of motorways such as radius and length of each curves and the number of lanes in both directions for each freeway section.

The ASPI segment database has been integrated with data gathered from Google Earth in order to collect missing data required for the analysis such as lane width, inside and outside shoulder width, median width, median and outside barriers.

Google Earth was used to measure the not available features such as cross sections elements and barriers extensions on a given motorway segment. An example of the methodology used to determine these geometrical features is illustrated in Figure 3.4.

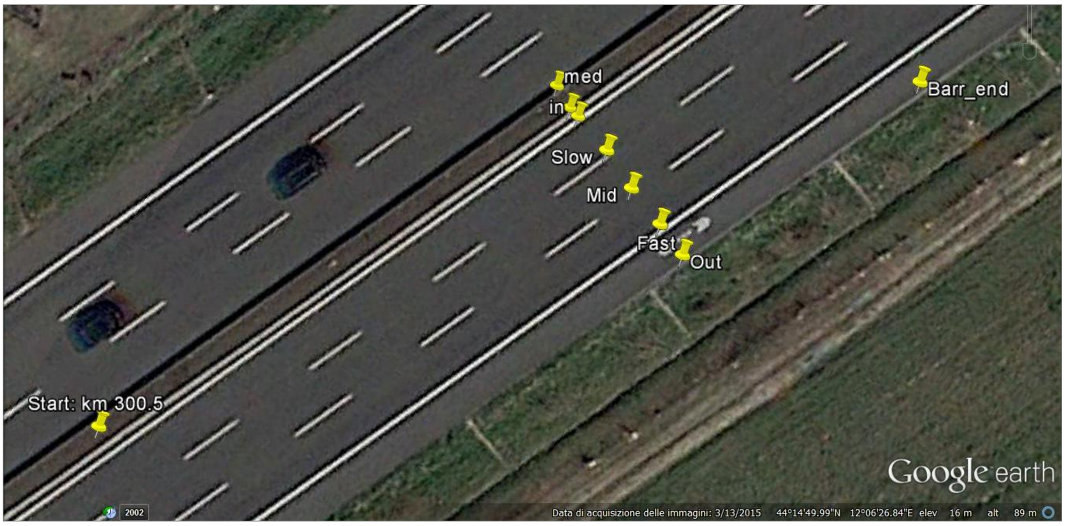



Figure 3.4: Example of segment features gathered from Google Earth

Google Earth software allowed to define the cross section elements (i.e., lanes width, inside and outside shoulder width, median width) by means of seven “placemarks” located along the cross section. Another set of “placemarks” allowed to determine the proportion of the segment length with a roadside safety barrier.

**Traffic database**

The traffic database contains the annual average daily traffic (AADT) for each motorway segment and direction (Figure 3.5). Traffic data available did not contain specific traffic counts within the work zones. Traffic usually tends to decrease during the presence of the work zone as the road users tend to choose alternative paths to reach their destination. However, this is generally not the case in motorways where alternative routes are usually not convenient, even if the work zone is installed.

| Motorway | Direcction | Segment Start (km) | Segment End (km) | Average AADT | AADT 2007 | AADT 2008 | AADT 2009 | AADT 2010 | AADT 2011 | AADT 2012 |
|---|---|---|---|---|---|---|---|---|---|---|
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |
| x | x | x | x | x | x | x | x | x | x | x |

Figure 3.5: Traffic Database

### *3.2.3 Descriptive analysis of work zone crashes*

In this section, a general overview of severe crashes occurred within the 30,389 work zones installed on Italian Motorways managed by ASPI between 2007 and 2012 is presented. Within the study period, 762 crashes with fatalities and/or injuries were recorded and resulted in 21 fatalities and 1,252 injuries. Of these accidents, 536 (70.3%) were multi-vehicle (MV) and 226 (29.7%) were single-vehicle (SV) crashes (Figure 3.6).

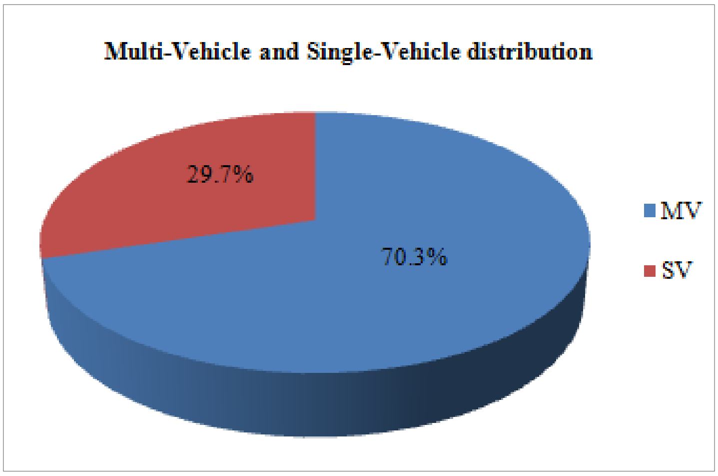



Figure 3.6: Work zone crash type distribution

**Distribution of work zone crashes over the years**

Figure 3.7 shows the distribution of severe crashes over the years of the analysis period. The histogram highlights a strong increase in the total number of accidents in the year 2009 as compared to the year 2008, where only 9% of the total number of work zone crashes was recorded. The increasing trend is also confirmed during the year 2010. Then, a slight decrease in crash rates is recorded in the following years 2011 and 2012.

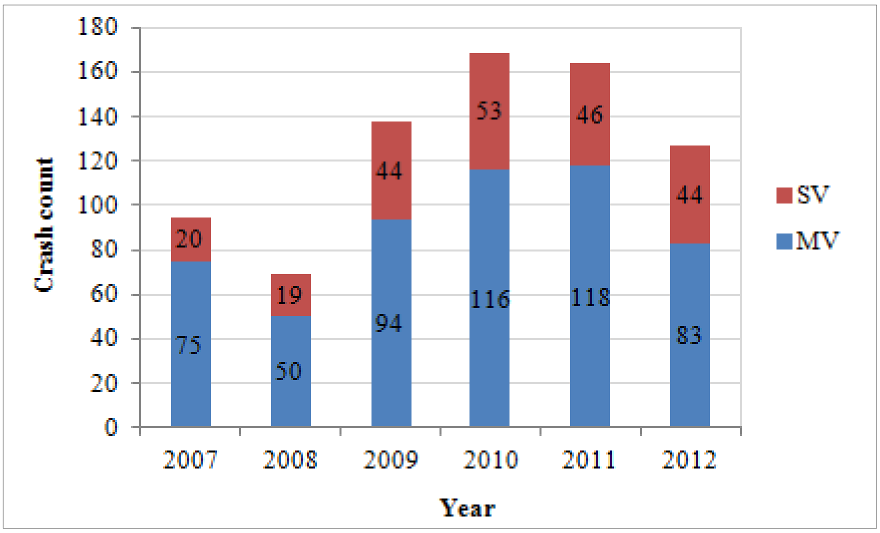



Figure 3.7: Crash distribution over the years

This trend can be however considered relevant only if compared to the number of work zones, their temporal duration and spatial extension.

Figure 3.8 shows the proportion of severe crashes occurred over the years compared to the proportion of work zone installed.

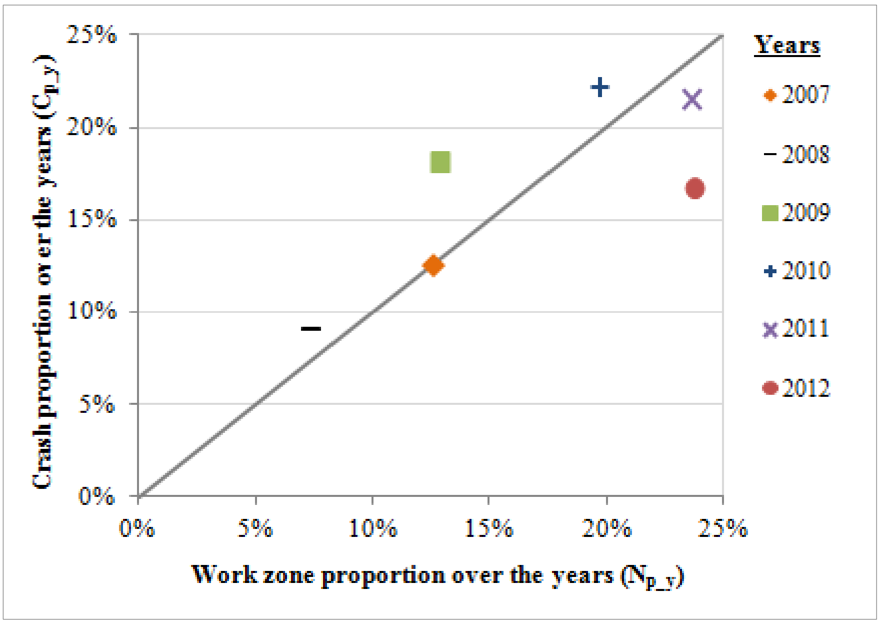



Figure 3.8: Crash proportion as compared to the proportion work zones over the years

In this chart the proportion of work zones for each year is defined as:

$$N_{p\_y} = \frac{\sum_{i=1}^{ny} L_i \cdot D_i}{\sum_{y=2007}^{2012} \sum_{i=1}^{ny} L_i \cdot D_i} \cdot 100 \qquad (3.1)$$

The proportion of crashes that occurred in the year “y” is defined as:

$$C_{p\_y} = \frac{c_y}{\sum_{y=2007}^{2012} c_y} \cdot 100 \tag{3.2}$$

Where:

- $n_y$ is the number of work zones in the year “y”;
- $c_y$ is the number of work zone crashes occurred in the year “y”;
- L is the length of the work zone “i”;
- D is the duration of the work zone “i”.

The results show that in 2008, despite a much lower crash rate compared to that recorded in other years, a higher share of crashes compared to the share of work zones is observed. On the other hand, despite the relatively high crash rate, the years 2011 and 2012 show lower crash proportions compared to the share of work zones. About 48% of all roadworks occurred in the 2-years period from 2011 to 2012, and were associated to only 38% of the overall number of crashes recorded from 2007 to 2012.

These findings suggest an overall improvement, on work zone traffic safety, over the last two years of the analysis period (2011-2012), probably due to factors such as the implementation of effective traffic control devices within the work zone areas, new policies or increased enforcement.

**Crash type distributions**

Figure 3.9 shows the distribution of severe crashes by crash type. The types with percentages of 2% or less were combined together and categorized as “others”. As shown in the pie chart in Figure 3.10, the most frequent type of crash is rear-end (48.8%) and rear-end, run-off and side collisions are the leading types of fatal and injury work zone crashes in Italy covering 77% of severe crashes in work zones. The dominance of rear-end collisions in severe crashes confirms the results of previous research. As discussed previously, rear-end crashes are mainly caused by vehicles driving at different speeds, resulting in a high speed variance. In addition, the higher proportion of multi-vehicle crashes indicated a higher interaction of vehicles within work zones, which can be attributed to the high speed variances.

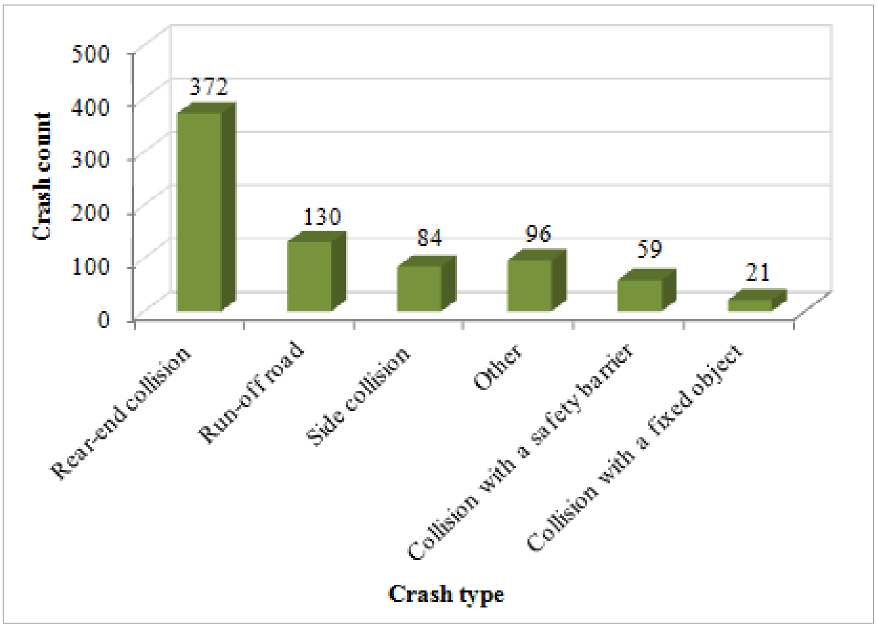



Figure 3.9: Impact of crash types on the frequency of fatal + injury crashes

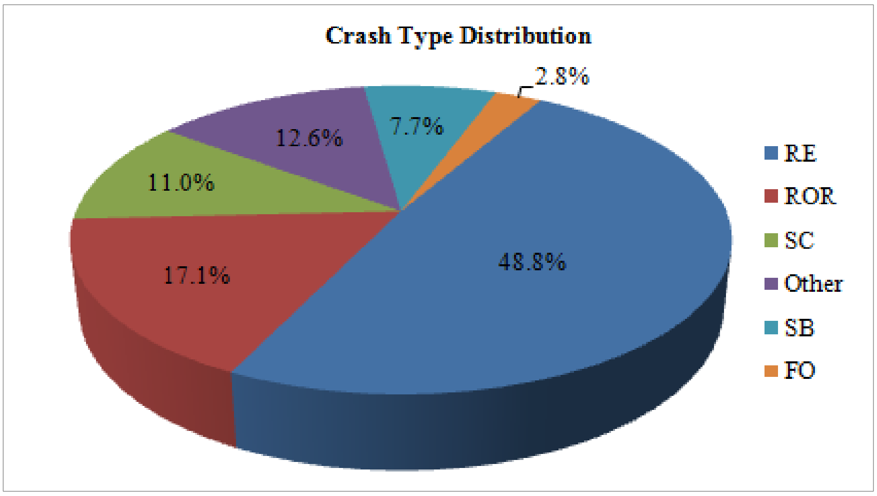



Figure 3.10: Percentage distribution of crash types

**Distribution of crashes according to the time of day**

Out of the 762 severe accidents that occurred in work zone areas from 2007 to 2012, 555 (72.8%) were daytime crashes and 207 (27.2%) were nighttime crashes. Daytime and nighttime conditions have been defined on the basis of the civil twilight[2]. Civil twilight is approximately the period of day in which solar illumination is sufficient, under clear weather conditions, to clearly distinguish terrestrial objects and, usually

[2] Twilight is defined according to the solar elevation angle, which is the position of the geometric centre of the sun relative to the horizon. Morning civil twilight begins when the geometric centre of the sun is 6° below the horizon (civil dawn), and ends at sunrise or when the geometric centre of the sun is 0°50′ below the horizon. Evening civil twilight begins at sunset or when the geometric centre of the sun is 0°50′ below the horizon, and ends when the geometric centre of the sun reaches 6° below the horizon (civil dusk) (USNO, 2016).

coincides with the period of day in which artificial lighting is switched on. As shown in Figure 3.11, fatal work zone crashes were more likely to be at nighttime, as compared to injury crashes. Out of a total of 21 fatalities recorded in work zones, 8 (38.1%) fatalities occurred at daytime and 13 (61.9%) at nighttime.

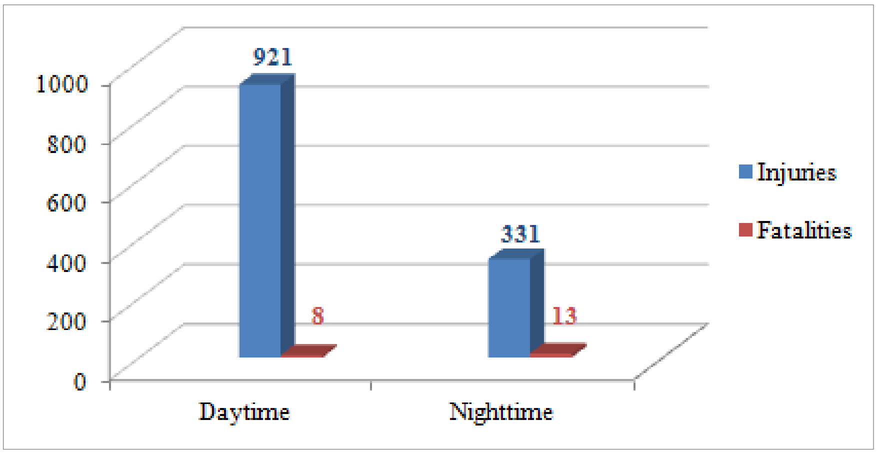



Figure 3.11: Number of fatalities and injuries by time of day

The distribution of different crash types as a function of the time of day is reported Figure 3.12.

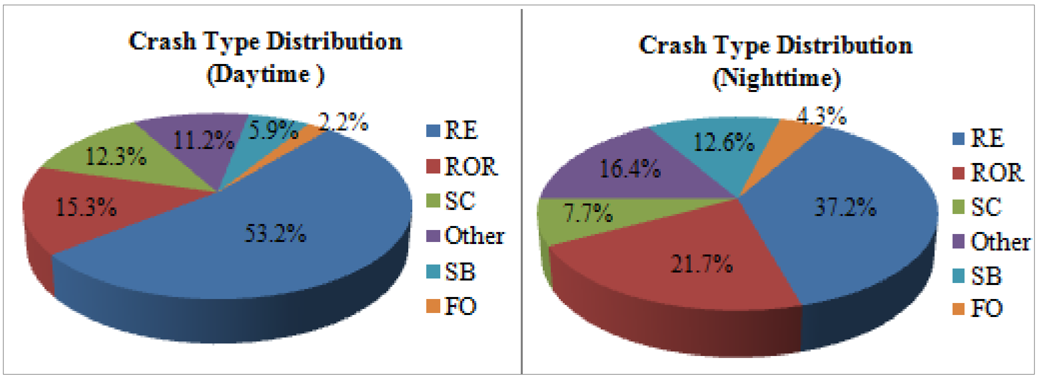



Figure 3.12: Crash type distributions during daytime (left) and nighttime (right)

The Pearson chi-square independence test has been performed to check whether the differences in crash type distributions, between daytime and nighttime, were statistically significant. SPSS software package was used to perform the independence test. The procedure is described in Appendix B.

In Table 3.3 the crash counts present subscript letters indicating the results of the z-test for the comparison of columns proportions and the different subscripts indicate that these proportions are statistically different at level of significance of 5%.

Table 3.3: Crash types distribution during daytime and nighttime

Type of collision*Time of day Crosstabulation

| | | | Time of day | | |
|---|---|---|---|---|---|
| | | | Daytime | Nighttime | Total |
| Type of collision | RE | Count | $295_a$ | $77_b$ | 372 |
| | | % within Time of day | 53.2% | 37.2% | 48.8% |
| | ROR | Count | $85_a$ | $45_b$ | 130 |
| | | % within Time of day | 15.3% | 21.7% | 17.1% |
| | SC | Count | $68_a$ | $16_a$ | 84 |
| | | % within Time of day | 12.3% | 7.7% | 11.0% |
| | Other | Count | $62_a$ | $34_b$ | 96 |
| | | % within Time of day | 11.2% | 16.4% | 12.6% |
| | SB | Count | $33_a$ | $26_b$ | 59 |
| | | % within Time of day | 5.9% | 12.6% | 7.7% |
| | FO | Count | $12_a$ | $9_a$ | 21 |
| | | % within Time of day | 2.2% | 4.3% | 2.8% |
| Total | | Count | 555 | 207 | 762 |
| | | % within Time of day | 100.0% | 100.0% | 100.0% |

Each subscript letter denotes a subset of Time of day categories whose column proportions do not differ significantly from each other at the .05 level.

As expected, the proportion of rear-end collisions during daytime (53.2%) is significantly higher than the proportion of rear-end at nighttime (37.2%), thus confirming the findings of past researches (Dimitropoulos et al., 1998; Ullman et al., 2008). The proportion of run-off road crashes (21.7%) significantly increases during nighttime as compared to daytime condition (15.3%), even though rear-end collision is still the dominant type.

Rear-end collisions are likely to be exacerbated by traffic congestion and queues associated with an increase in the AADT. Unstable traffic flow conditions may cause abrupt speed changes and therefore increase the chance of rear-end collisions.

Traffic volumes at night are typically much lower than during daytime hours. For this reason roadworks are often performed at night as lower volumes reduce vehicular exposure to the work zone. However, lower volumes provide greater freedom to manoeuvre to drivers, and allow higher operating speeds. Although night work is associated with a lower percentage, nighttime crashes are typically more severe because of generally higher traffic speeds.

### *3.2.4 Sampling and data reduction*

The work zone database provides information on the road works installed in the motorway network in the 6-year period from 2007 to 2012.

In this analysis only the stationary work zones installed along the motorway sections were extracted from the database, therefore excluding the work zones installed on the speed change lanes and on the motorway interchanges and their possible extensions on adjacent motorway segments. This choice is motivated by the significant differences between the interchanges and the segments in terms of road geometric design, speed limits, number of lanes and therefore potential effect of the work zone. Furthermore only the work zones installed on motorway segments whose geometric and functional characteristics were known, were used for the analysis. In order to have at least one year of observations for each work zone in the before period, only the work zones started after 1$^{st}$ January 2008 were included in the sample.

As a result of these considerations, the study was then carried out on a sample of 15,570 work zones. Table 3.4 provides the summary statistics for the work zone dataset considered split by layout configuration. For each layout the number, the total length, the total duration and the number of crashes occurred in the specific layout are given.

Table 3.4: Summary statistics for Italian work zone data

| **Layout** | **Number** | **Total length (km)** | **Total Duration (days)** | **Total fatal+injury crashes** | **SV fatal+injury crashes** | **MV fatal+injury crashes** |
|---|---|---|---|---|---|---|
| Slow2 | 3,945 | 7,283.31 | 5,904.65 | 20 | 6 | 14 |
| Fast2 | 2,964 | 6,912.40 | 7,038.14 | 21 | 6 | 15 |
| Emergency2 | 2,147 | 6,187.93 | 24,084.75 | 73 | 36 | 37 |
| Cross2(0+1) | 1,591 | 5,792.51 | 4,429.93 | 56 | 14 | 42 |
| Fast2(2) | 101 | 244.20 | 5,663.16 | 60 | 13 | 47 |
| Cross2(1+1) | 47 | 157.96 | 329.60 | 8 | 1 | 7 |
| Slow3 | 1,669 | 3,268.66 | 6,454.73 | 31 | 14 | 17 |
| Emergency3 | 1,233 | 5,742.02 | 16,190.52 | 77 | 21 | 56 |
| Fast3 | 915 | 2,869.29 | 3,254.66 | 23 | 3 | 20 |
| Slow&Middle3 | 406 | 931.07 | 342.98 | 4 | 1 | 3 |
| Middle&Fast3 | 290 | 764.65 | 235.65 | 3 | 1 | 2 |
| Cross3(0+1) | 108 | 336.02 | 258.30 | 5 | 1 | 4 |
| Cross3(1+1) | 79 | 329.21 | 215.90 | 9 | 1 | 8 |
| Middle&Fast3(2) | 41 | 79.00 | 3,064.79 | 14 | 7 | 7 |
| Fast3(3) | 24 | 84.65 | 648.74 | 14 | 5 | 9 |
| Cross3(0+2) | 10 | 34.30 | 1,567.82 | 16 | 2 | 14 |
| **All** | **15,570** | **41,017.18** | **79,684** | **434** | **132** | **302** |

In Figure 3.13 the average duration and length of the different layout configurations are shown: the average work zone duration is 5 days and the average length is 2.60 km.

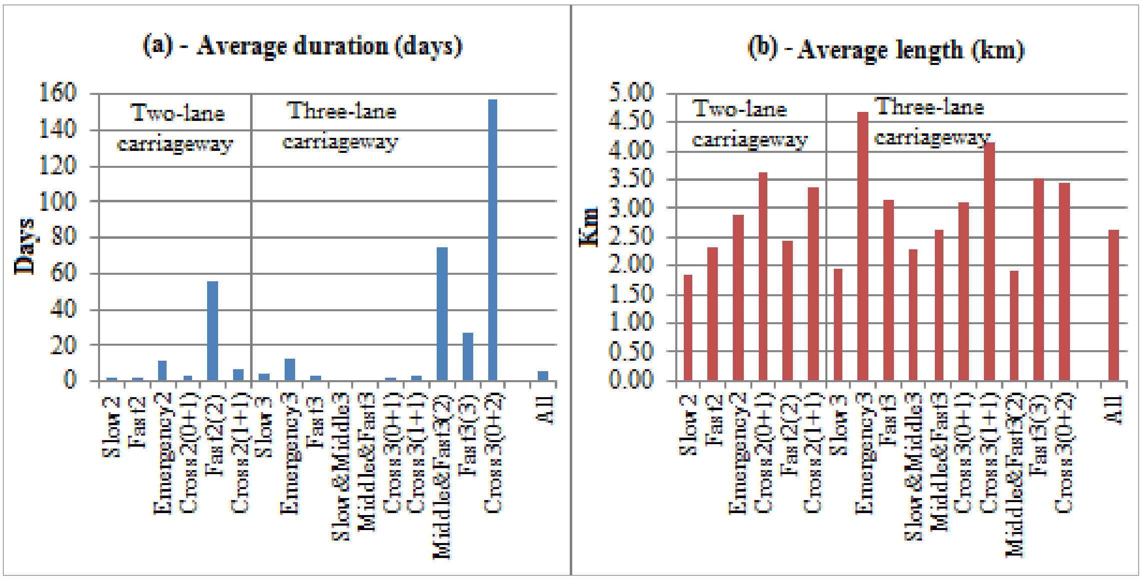



Figure 3.13: Average length and duration of different work zones

Crash and work zone data were then combined in order to define the number of crashes observed in each work zone. Due to the fact that many work zones crashes were not coded as involving a work zone, the projects' limits were then extended to 0.8 km before and 0.5 km after the activity area, in order to account for the advance warning area and for the termination area. Therefore the sections in approach to the activity area, where warning signs and possible traffic queues might affect the driver's behaviour and crash propensity, were included in the analysis as "work zone areas". Then, a thorough check of databases has been conducted, in collaboration with the road administration, in order to understand if the crashes effectively occurred within a work zone.

Figure 3.14 shows the proportion of crashes occurred in each work zone configuration as compared to the proportion of work zone that have been installed with that given configuration. This highlights that some layouts ("Cross2(1+1)", "Cross3(1+1)", "Cross3(0+1)", "Cross2(0+1)", "Slow&Middle3", "Middle&Fast3", "Fast2(2)", "Fast3(3)", "Fast3", "Emergency3") have a share of crashes higher than the share of work zones.

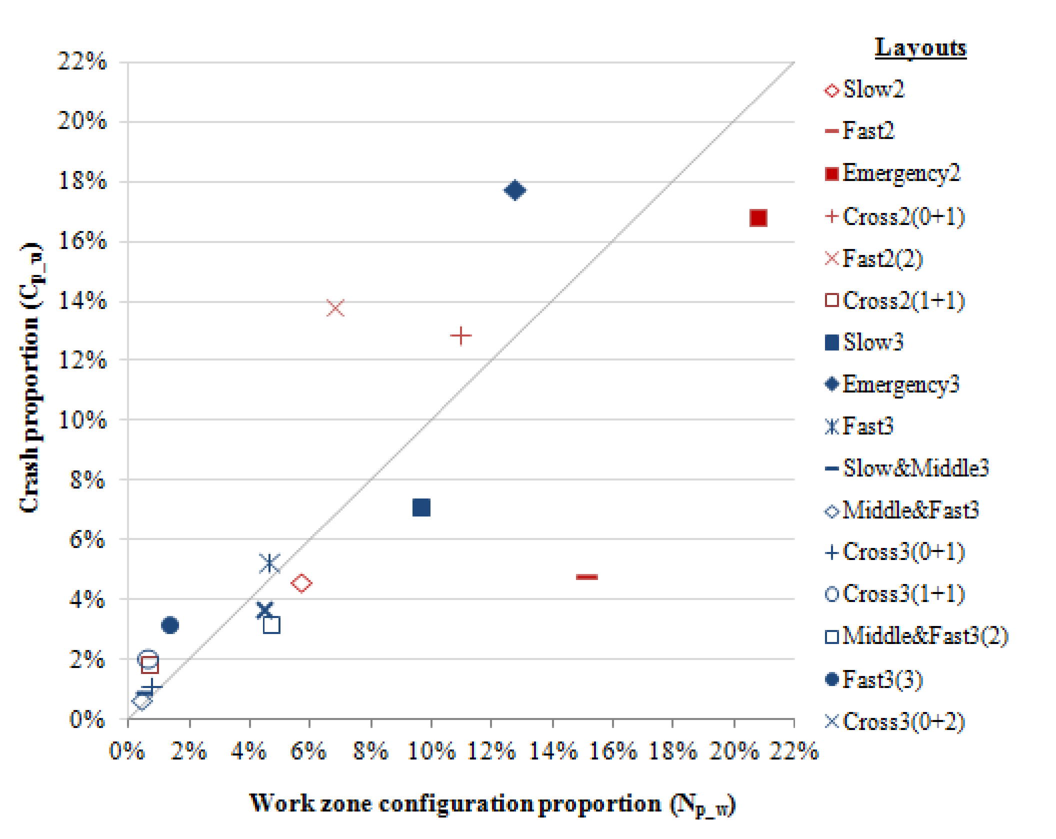



Figure 3.14: Crash proportion as compared to the proportion of each work zone layout

In this chart the proportion of each layout configuration is computed with equation (3.1). The proportion of crashes that occurred in a given layout configuration "w" is defined as:

$$C_{p\_w} = \frac{\sum_{i=1}^{nw} c_i}{\sum_{w=1}^{nl} \sum_{i=1}^{nw} c_i} \cdot 100 \tag{3.3}$$

Where:

- $n_w$ is the number of work zones with a given layout configuration "w";
- $n_l$ is the number of the different layout configurations;
- $c_i$ is the number of crashes occurred within the work zone "i";
- L is the length of the work zone "i";
- D is the duration of the work zone "i".

## 3.3 Methodology

The EB method (Hauer, 1997) was used in this study in order to estimate the number of crashes that could be expected if the work zone had not occurred at a specific site. The EB method allows us to make reliable estimations of the expected crashes because it takes into consideration both the systematic variations and the partially random fluctuations of crashes. The latter causes the well known statistical phenomenon called Regression To the Mean (RTM) bias. The EB method properly accounts for RTM effect and determines a smoothed value for expected crashes in order to eliminate typical errors associated with crash counts (time trends as well as RTM bias).

The intent of the EB procedure is to estimate the expected number of crashes that would have occurred within a road segment if the work zone had not been installed. The expected number of crashes to be compared to the number of reported crashes during the work zone activities over the same period of interest is estimated by combining the prediction from the predictive model with the observed crash data by using the EB method. The predictive model used in this study is described in detail in the following paragraph.

### *3.3.1 Segmentation process*

The analysis of the crash database highlighted that part of records clustered in correspondence of the freeway mileposts. This is probably due to some inaccuracies in the crash reports where the accidents are sometimes assigned to the freeway milepost and not to the exact location. In order to overcome this problem of clustering of crashes the freeway network has been divided into one km long segments, with their centre in the milepost (the segment starts at km “i” +500 km and ends at km “i+1”+500).

The first and the last segment of each freeway section, defined as a portion of freeway between two interchanges, are therefore always shorter than 1 km. The HSM recommends a minimum segment length of 0.16 km. Shorter roadway segments are undesirable because the segment characteristics may not be in place for sufficient length to truly affect crash risk and because data on crash locations may not be accurate enough to assign each crash to the appropriate road segment. Therefore segments shorter than 0.16 km were excluded from the analysis as shown in Figure 3.15.

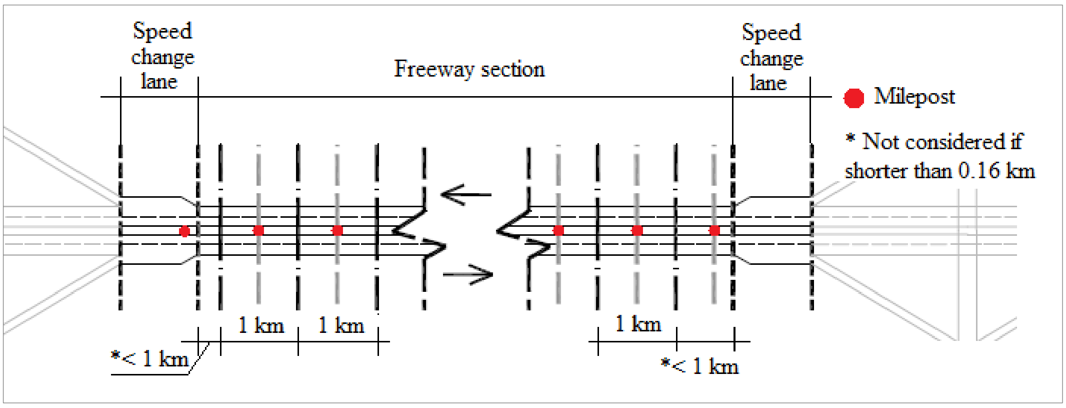



Figure 3.15: Segmentation process

This segmentation methodology has been applied also considering the limited variability of the geometric features that influence the HSM segmentation process in each section.

### *3.3.2 Crash Prediction Model*

A Crash Prediction Model is a mathematical equation used to predict the crash experience taking care of the systematic variations of crashes induced in the given site, by its traffic and physical characteristics.

For the evaluation of the expected crash frequencies the model proposed in the NCHRP 17-45 project (Bonneson et. al., 2012) and published in the Highway Safety Manual Supplement (AASHTO, 2014) has been adopted. The predictive models used to determine the predicted average crash frequency are of the general form shown in (3.4), as in the HSM models (AASHTO, 2010).

The method uses three components to predict the average expected crash frequency at a site:

- the base model, called a Safety Performance Function (SPF);
- the Crash Modification Factors (CMFs) to adjust the estimate for additional site specific conditions, that may be different from the base conditions;
- a calibration factor to adjust the estimate for accuracy in local conditions.

These components are combined in the form below:

$$N_{PRED} = N_{SPF} \cdot (CMF_1 \cdot CMF_2 \cdot ...... \cdot CMF_m) \cdot C \qquad (3.4)$$

Where:

- $N_{PRED}$ is the predicted average crash frequency of the freeway segment (crashes/yr);

- $N_{SPF}$ is the predicted average crash frequency determined for base conditions of the SPF developed for a freeway segment (crashes/yr);
- $CMF_m$ is the crash modification factor for the design feature $m$;
- $C$ is the calibration factor to adjust SPF for local conditions of the freeway segment.

The predictive model for freeway segments is composed by two terms, each of them providing the estimated crash frequency for a specific crash type and severity (single vehicle crashes with fatal and injury, multiple vehicle crashes with fatal and injury).

### *3.3.3 Safety Performance Functions*

When using the predictive method, the appropriate safety performance functions (SPFs) are used to estimate the predicted average crash frequency of a site with base conditions in terms of design features and operating conditions.

Eight SPFs for fatal + injury crashes on freeway segments have been used for this analysis. The SPFs are summarized in Table 3.5.

Table 3.5: Freeway Safety Performance Functions used in the analysis

| Number of Through Lanes (n) | Area Type | Crash Type ($y$) |
|---|---|---|
| 4 | Rural | Multiple vehicle |
| 4 | Rural | Single vehicle |
| 4 | Urban | Multiple vehicle |
| 4 | Urban | Single vehicle |
| 6 | Rural | Multiple vehicle |
| 6 | Rural | Single vehicle |
| 6 | Urban | Multiple vehicle |
| 6 | Urban | Single vehicle |

The base conditions for the SPFs for multi-vehicle and single-vehicle crashes on freeway segments are listed in Table 3.6 as function of a number of variables defined in Chapter 3.3.4.

Table 3.6: Base conditions for the SPFs (AASHTO, 2014)

| Variable | Base condition | |
|---|---|---|
| | **Multi-Vehicle crashes** | **Single-Vehicle crashes** |
| Length of the horizontal curve | not present | not present |
| Lane width | 3.66 m | 3.66 m |
| Inside shoulder width (paved) | 1.83 m | 1.83 m |
| Median width | 18.29 m | 18.29 m |
| Length of median barrier | not present | not present |
| Number of hours where volume exceeds 1,000 veh/h/ln | None | None |
| Distance to nearest upstream ramp entrances | More than 804.67 m from segment | - |
| Distance to nearest downstream ramp exits | More than 804.67 m from segment | - |
| Outside shoulder width (paved) | - | 3.05 m |
| Clear zone width | - | 9.14 m |
| Length of outside barrier | - | not present |

The general form for the SPF is given by the following equation:

$$N_{SPF} = L \cdot \exp[a + b \cdot \ln(c \cdot AADT)] \tag{3.4}$$

Where:

- $L$ is the length of freeway segment (mi);
- $AADT$ is the annual average daily traffic volume of freeway segment (veh/day);
- $a$, $b$ are regression coefficients;
- $c$ is a scale factor for AADT.

The SPF coefficients and the inverse dispersion parameter (K) for single vehicle (SV) and multi-vehicle (MV) crashes are listed in Table 3.7 and Table 3.8.

Table 3.7: SPF Coefficients for Single-Vehicle Fatal + Injury Crashes on Freeway Segments (AASHTO, 2014)

| Number of Through Lanes (n) | Area Type | SPF coefficients | | | Inverse dispersion Parameter K (mi-1) |
|---|---|---|---|---|---|
| | | **a** | **b** | **c** | |
| 4 | Rural | -2.126 | 0.646 | 0.001 | 30.1 |
| 6 | Rural | -2.055 | 0.646 | 0.001 | 30.1 |
| 4 | Urban | -2.126 | 0.646 | 0.001 | 30.1 |
| 6 | Urban | -2.055 | 0.646 | 0.001 | 30.1 |

Table 3.8: SPF Coefficients for Multi-Vehicle Fatal + Injury Crashes on Freeway Segments (AASHTO, 2014)

| Number of Through Lanes (n) | Area Type | SPF coefficients | | | Inverse dispersion Parameter K (mi-1) |
|---|---|---|---|---|---|
| | | a | b | c | |
| 4 | Rural | -5.975 | 1.492 | 0.001 | 17.6 |
| 6 | Rural | -6.092 | 1.492 | 0.001 | 17.6 |
| 4 | Urban | -5.470 | 1.492 | 0.001 | 17.6 |
| 6 | Urban | -5.587 | 1.492 | 0.001 | 17.6 |

Furthermore, HSM defines ranges of AADT volume for which these SPFs are applicable. These ranges of AADT volume for freeway segments with 4 and 6 through lanes (total of both travel directions) are shown in Table 3.9.

Table 3.9: Applicable AADT Volume Ranges for SPFs (AASHTO, 2014)

| Number of Through Lanes (n) | Area Type | Applicable AADT volume range (veh/day) |
|---|---|---|
| 4 | Rural | 0 to 73,000 |
| 6 | Rural | 0 to 130,000 |
| 4 | Urban | 0 to 110,000 |
| 6 | Urban | 0 to 180,000 |

Application of the SPFs to sites with AADT volumes substantially outside these ranges may not provide reliable results.

The SPFs used for the four-lane median divided motorways are plotted in Figure 3.16, whereas those used for the six-lane median divided motorways are plotted in Figure 3.17.

The value of the over-dispersion parameter (k) associated with SPFs for crash type *y* (single-vehicle, multi-vehicle) is determined as follows:

$$k_y = \frac{1}{K_y \cdot L} . \tag{3.5}$$

Where:

- $K_y$ is the inverse dispersion parameter of a freeway segment with crash type y (single-vehicle, multi-vehicle) (mi-1).

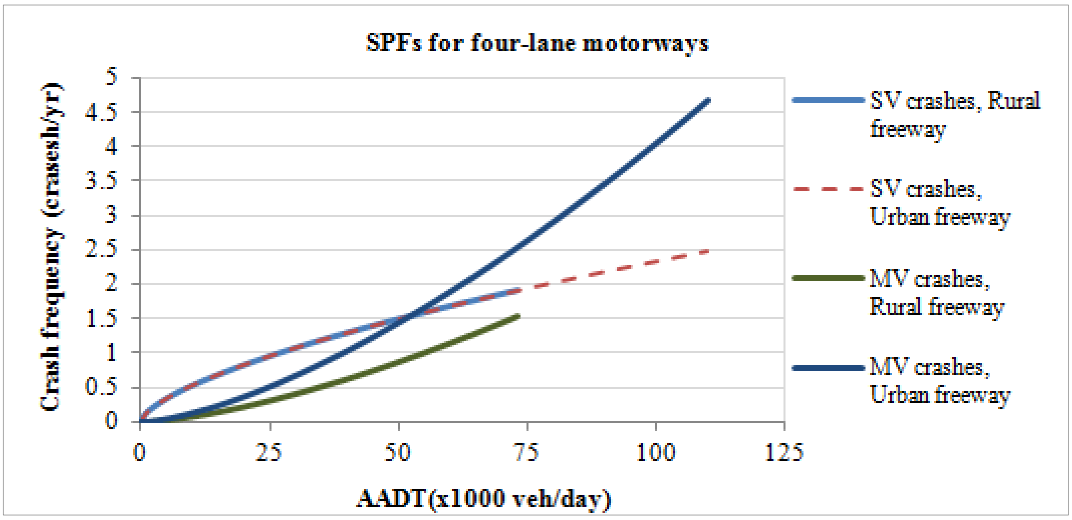



Figure 3.16: SPFs for the four-lane median divided motorways

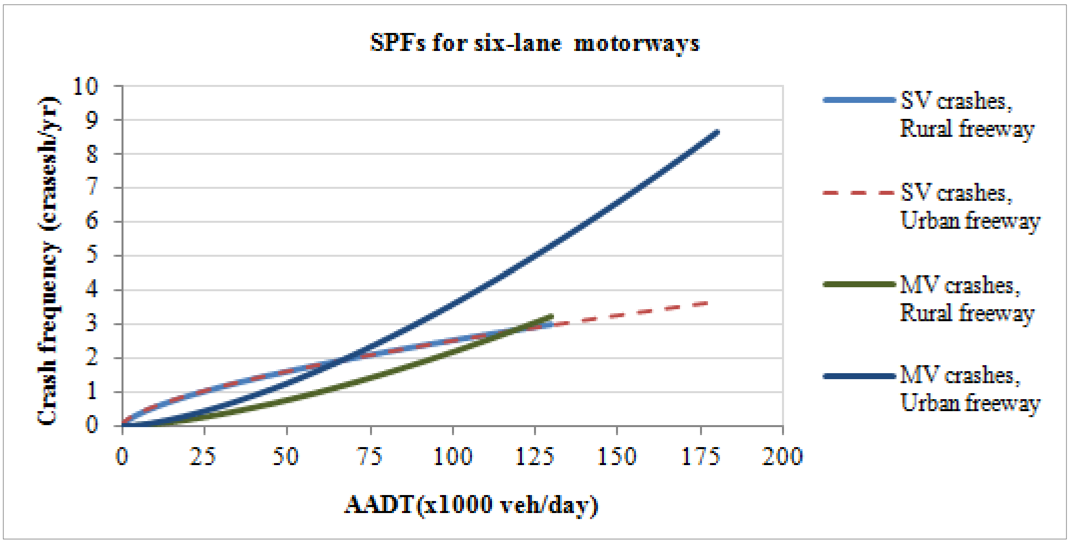



Figure 3.17: SPFs for the six-lane median divided motorways

### *3.3.4 Crash Modification Factors*

A set of CMFs are included in the model for several design features and they represent the estimated effects of a change in a given variable on the expected number of crashes. The variables considered in the prediction model are described below. Given the segmentation procedure adopted, some of the variables vary within the freeway segments. In such cases, an "equivalent" CMF has been computed as a length-weighted average of the different conditions measured along the segments.

**Horizontal curvature**

The CMFs describing the relationship between the horizontal curvature and the predicted crash frequency have been computed by using the following equation:

$$CMF_{1,y} = 1 + a \cdot \left[ \sum_{i=1}^{m} \left( \frac{5{,}730}{R_i^*} \right)^2 \cdot P_{c,i} \right] \tag{3.6}$$

Where:

- $CMF_{1,y}$ is the crash modification factor for horizontal curvature in a freeway segment as a function of the crash type y (single-vehicle, multi-vehicle);
- $R^*_i$ is the radius of curve $i$ (ft);
- $P_{c,i}$ is the proportion of segment length with curve $i$;
- $m$ is the number of horizontal curves in the segment.

The coefficients for fatal + injury crashes on freeway segments to be used in Equation (3.6) are provided in Table 3.10 as a function of the crash type (multi-vehicle and single-vehicle crashes).

Table 3.10: Coefficients for horizontal curvature for fatal + injury crashes on freeway segments (AASHTO, 2014)

| **Crash Type (y)** | **CMF Variable** | **CMF coefficient (a)** |
|---|---|---|
| Multiple vehicle | $CMF_{1,mv}$ | 0.0172 |
| Single vehicle | $CMF_{1,sv}$ | 0.0719 |

The CMF is applicable to curves with a radius larger than 304.8 m. The curve length is measured along the reference line, defined as the inside edge of carriageway for the roadbed serving traffic moving in the increasing milepost direction (Figure 3.18).

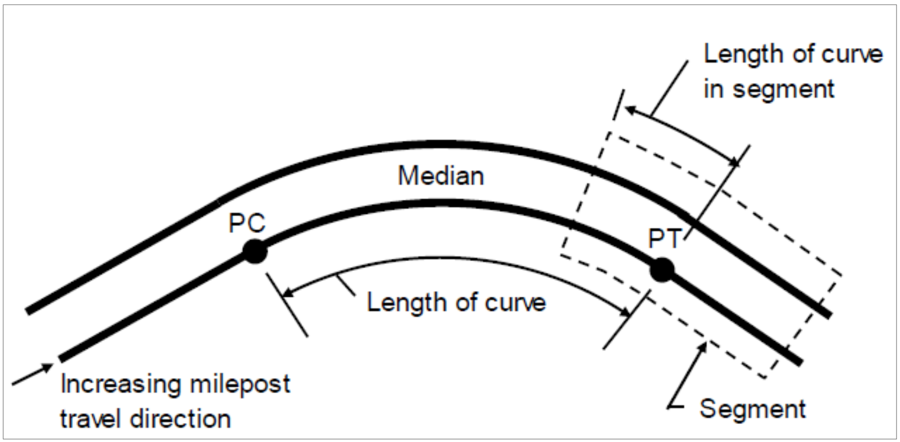



Figure 3.18: Curve length and radius measurements (HSM, 2014)

The curve radius is measured separately for each roadbed along the reference line and the length is that measured within the limits of the freeway segment.

**Lane width**

The CMFs describing the relationship between the average lane width and the predicted crash frequency have been computed by using the following equations:

$$CMF_2 = \begin{cases} \exp[-0.0376 \cdot (W_l - 12)] & if\ W_l < 13ft \\ 0.963 & if\ W_l \geq 13ft \end{cases} \tag{3.7}$$

Where:

- $CMF_2$ is the crash modification factor for lane width in a freeway segment for fatal + injury single-vehicle and multi-vehicle crashes;
- $W_l$ is the average lane width (ft).

The lane width has been computed as an average for all through lanes and where the width varied along the freeway segment, the length-weighted average width has been computed for that segment.

The CMFs values are the same for both multi-vehicle and single-vehicle crashes and are plotted in Figure 3.19. The CMF is discontinuous, breaking at a lane width of 3.96 m and applicable to lane widths ranging between 3.20 m and 4.27 m.

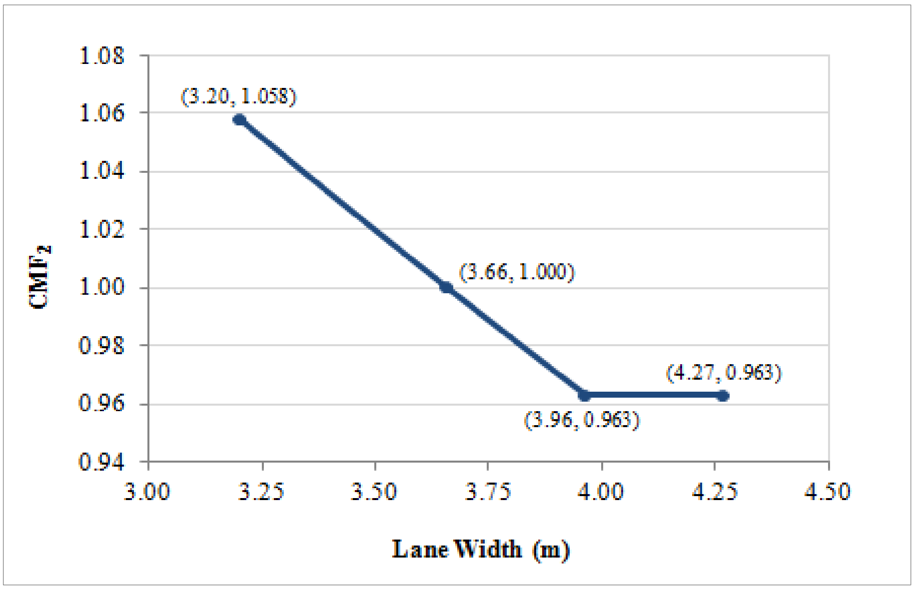



Figure 3.19: Relationship between average lane width and CMF value

**Inside shoulder width**

The CMFs describing the relationship between the average inside shoulder width and the predicted crash frequency have been computed by using the following equation:

$$CMF_3 = \exp[-0.0172 \cdot (W_{is} - 6)] \tag{3.8}$$

Where:

- $CMF_3$ is the crash modification factor for the average inside shoulder width of a freeway segment for fatal + injury single-vehicle and multi-vehicle crashes;
- $W_{is}$ is the paved inside shoulder width (ft).

The CMFs values are the same for both multi-vehicle and single-vehicle crashes and are plotted in (Figure 3.20). The CMF is applicable to shoulder widths ranging from 0.61 m to 3.66 m.

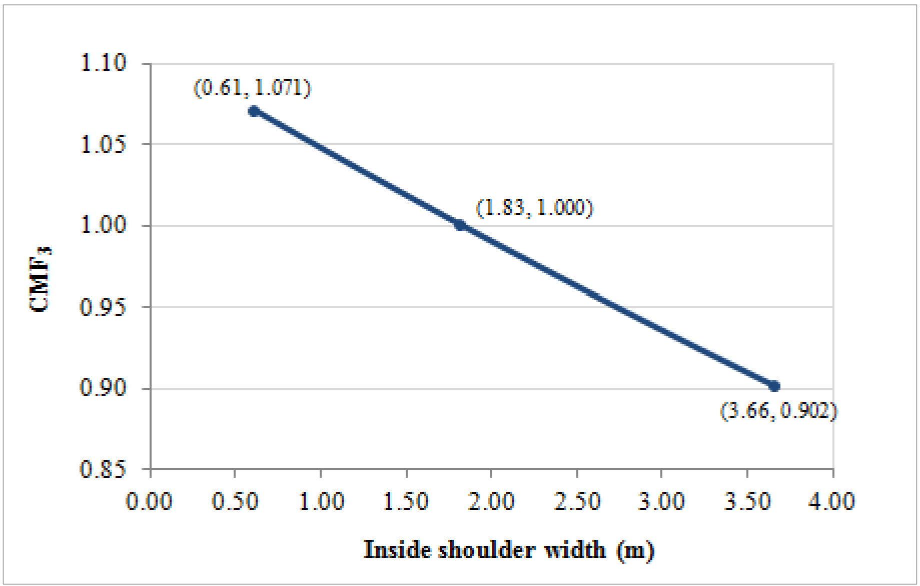



Figure 3.20: Relationship between average inside shoulder width and CMF value

**Median barrier**

The CMFs describing the relationship between the median barrier presence and the predicted crash frequency have been computed by using the following equation:

$$CMF_4 = (1 - P_{ib}) + P_{ib} \cdot \exp\left(\frac{0.131}{W_{icb}}\right) \quad (3.9)$$

Where:

- $CMF_4$ is the crash modification factor for median barrier presence in a freeway segment for fatal + injury single-vehicle and multi-vehicle crashes;
- $P_{ib}$ is proportion of segment length with a safety barrier present in the median;
- $W_{icb}$ is the distance from edge of the paved inside shoulder to barrier face (ft).

Openings of the median were not considered as they are usually protected by movable barriers on the Italian motorways: as a consequence of this consideration the $P_{ib}$ value is always set to 1.

HSM recommends values of $W_{icb}$ ranging from 0.23 m to 5.18 m as the applicability domain of the CMF. The distance from the edge of the inside shoulder to barrier face is typically equal to zero in Italian motorways. However the CMF increases unrealistically for $W_{icb}$ values lower than 0.23 m as shown Figure 3.21. The minimum value of 0.23 m was therefore used for the calculations, corresponding to a CMF value equal to 1.191 for both fatal + injury single-vehicle and multi-vehicle crashes.

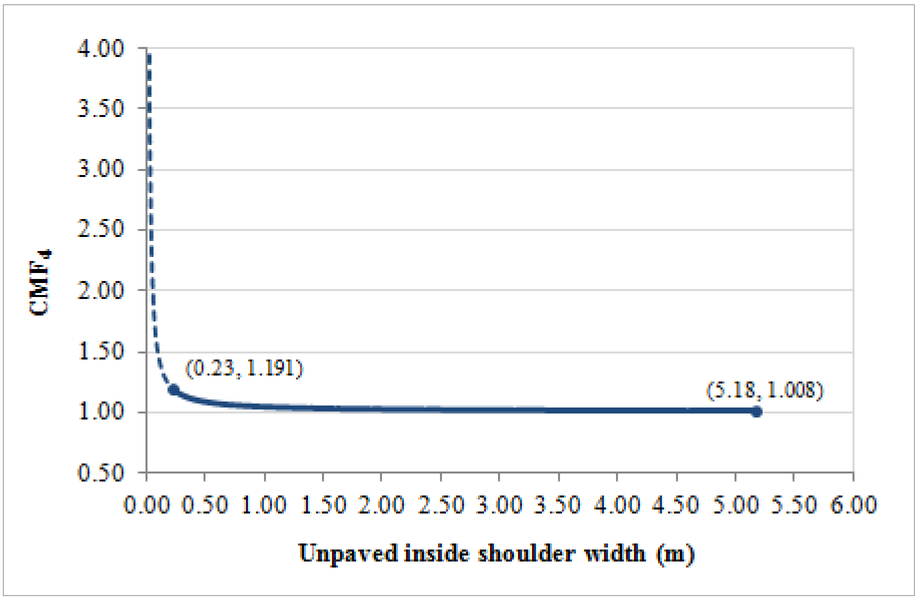



Figure 3.21: Relationship between unpaved inside shoulder width and CMF value

**Median width**

The CMFs describing the relationship between the average median width and the predicted crash frequency have been computed by using the following equation:

$$CMF_{5,y} = (1 - P_{ib}) \cdot \exp[a \cdot (W_m - 2 \cdot W_{is} - 48)] + P_{ib} \cdot \exp(-46.5 \cdot a) \qquad (3.10)$$

Where:

- $CMF_{5,y}$ is the crash modification factor for median width of a freeway segment as a function of the crash type y (single-vehicle, multi-vehicle);
- $W_m$ is the median width (measured from near edges of the opposing carriageway (ft).

Figure 3.22 illustrates the elements composing the median width.

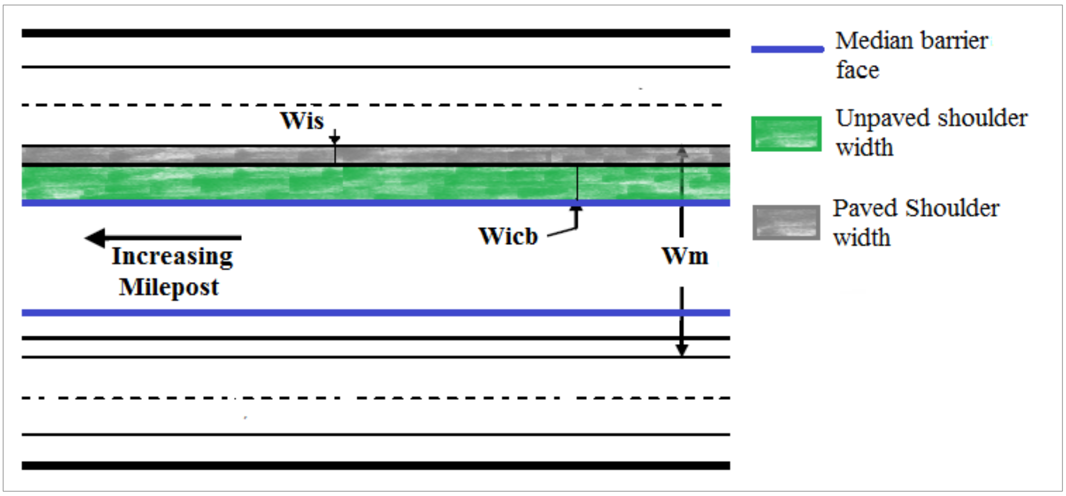



Figure 3.22: Elements within the median width

The coefficients for fatal + injury crashes on freeway segments to be used in Equation (3.10) are provided in Table 3.11 as a function of the crash type (multi-vehicle and single-vehicle crashes).

Table 3.11: Coefficients for median width for fatal + injury crashes on freeway segments (AASHTO, 2014)

| **Crash Type (y)** | **CMF Variable** | **CMF coefficient (a)** |
|---|---|---|
| Multiple vehicle | $CMF_{5,mv}$ | -0.00302 |
| Single vehicle | $CMF_{5,sv}$ | 0.00102 |

The sign of the coefficients in Table 3.11 indicates that multi-vehicle fatal + injury crash frequency decreases with an increase in median width and that single-vehicle fatal + injury crash frequency increases slightly with an increase in median width. This latter trend indicates that an errant vehicle is more likely to have a single-vehicle crash with a wide median and a multi-vehicle crash with a narrow median.

Considering the hypothesis of median barrier on the entire motorway segment ($P_{ib}$ = 1) and a constant value of the $W_{icb}$ equal to 0.23 m, the CMFs values are constant and equal to 1.151 and 0.954 respectively for fatal + injury multi-vehicle and single-vehicle crashes.

**High Volume**

The volume-to-capacity ratio relates the traffic demand to the capacity of a roadway segment. As volume nears capacity, average speed tends to decrease and headway is reduced. These changes have some influence on crash characteristics, including crash frequency and crash type distribution (i.e., single-vehicle and multi-vehicle).

The CMFs describing the relationship between the traffic concentration during peak hours and the predicted crash frequency have been computed by using the following equation:

$$CMF_{6,y} = \exp(a \cdot P_{hv}) \tag{3.11}$$

Where:

- $CMF_{6,y}$ is the crash modification factor for high volume in a freeway segment as a function of the crash type y (single-vehicle, multi-vehicle);
- $P_{hv}$ is the proportion of AADT during hours where volume exceeds 1,000 vehicles per hour per lane (veh/h/ln).

The coefficients for fatal + injury crashes on freeway segments to be used in Equation (3.11) are provided in Table 3.12 as a function of the crash type (multi-vehicle and single-vehicle crashes).

Table 3.12: Coefficients for high volume for fatal + injury crashes on freeway segments

| **Crash Type (y)** | **CMF Variable** | **CMF coefficient (a)** |
|---|---|---|
| Multiple vehicle | $CMF_{6,mv}$ | 0.350 |
| Single vehicle | $CMF_{6,sv}$ | -0.0675 |

The hourly volume (HV) per lane has been computed by using the mean value for the six years of analysis (2007-2012) for each motorway segment. The desired proportion $P_{hv}$ is then computed for each segment as follows:

$$P_{hv} = \frac{\sum_{i=1}^{24} HV_i^*}{AADT} \tag{3.12}$$

Where $HV_i^*$ is the traffic volume during hour *i* (*i* = 1, 2, 3,..., 24) where the lane volume exceeds 1,000 veh/h/ln.

The CMFs values as a function of $P_{hv}$ are plotted in Figure 3.23 for multi-vehicle and single-vehicle crashes.

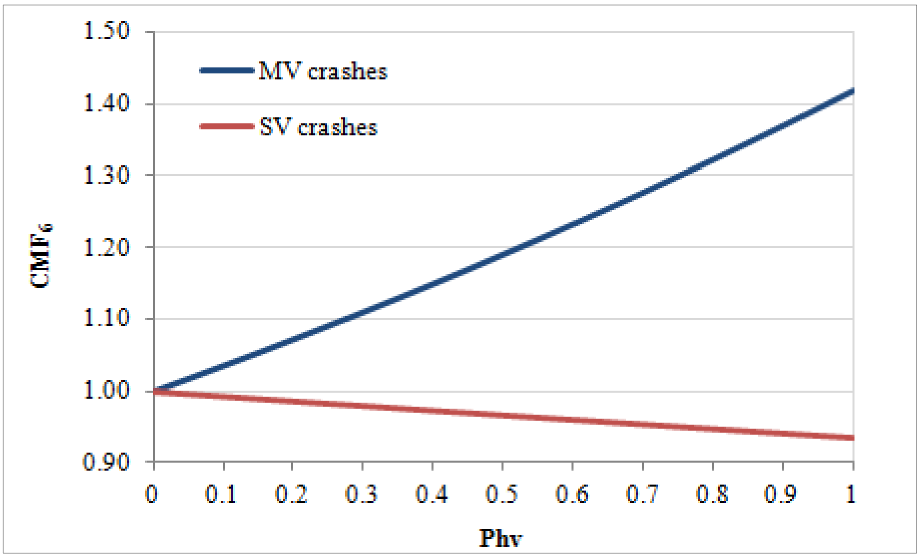



Figure 3.23: Relationship between the proportion of AADT during hours where volume exceeds 1,000 veh/h/ln and CMF value

**Lane change**

The presence of entrance or exit ramps creates a large number of lane changes on the freeway and a notable variation in lane volume. This CMF takes into account the influence of the interchange ramp presence on freeway crash frequency. The base condition (CMF = 1) is no entrance or ramp exit within a distance of 0.8 km from the segment.

The CMFs describing the relationship between the lane change activity and the predicted crash frequency have been computed by using the following equation:

$$CMF_{7,mv} = \left(1+\frac{\exp[-12.56\cdot X_{b,ent}-0.272\cdot\ln(0.001\cdot AADT_{b,ent})]}{12.56\cdot L}\cdot[1-\exp(-12.56\cdot L)]\right)\cdot \left(1+\frac{\exp[-12.56\cdot X_{e,ext}-0.272\cdot\ln(0.001\cdot AADT_{e,ext})]}{12.56\cdot L}\cdot[1-\exp(-12.56\cdot L)]\right) \quad (3.13)$$

Where:

- $CMF_{7,mv}$ is the crash modification factor for lane changes in a freeway segment for fatal + injury multi-vehicle crashes;
- $X_{b,ent}$ is the distance from segment begin to nearest upstream entrance ramp gore point (mi);
- $X_{e,ext}$ is the distance from segment end milepost to nearest downstream exit ramp gore point(mi);

- $AADT_{b,ent}$ is the AADT volume of entrance ramp located at distance $X_{b,ent}$ (veh/day);
- $AADT_{e,ext}$ is the AADT volume of exit ramp located at distance $X_{e,ext}$ (veh/day).

Distance to nearest upstream and downstream ramps is measured from the segment boundary to the ramp gore point, along the freeway's white pavement edge marking that intersects the gore point (Figure 3.24).

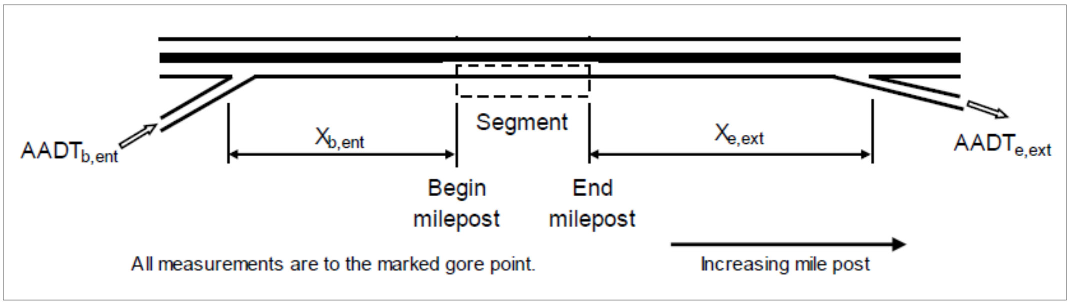



Figure 3.24: Distance to nearest ramps (AASHTO, 2014)

The gore point is located where the pair of white pavement edge markings that separate the ramp from the freeway main lanes are 0.60 m (Figure 3.25).

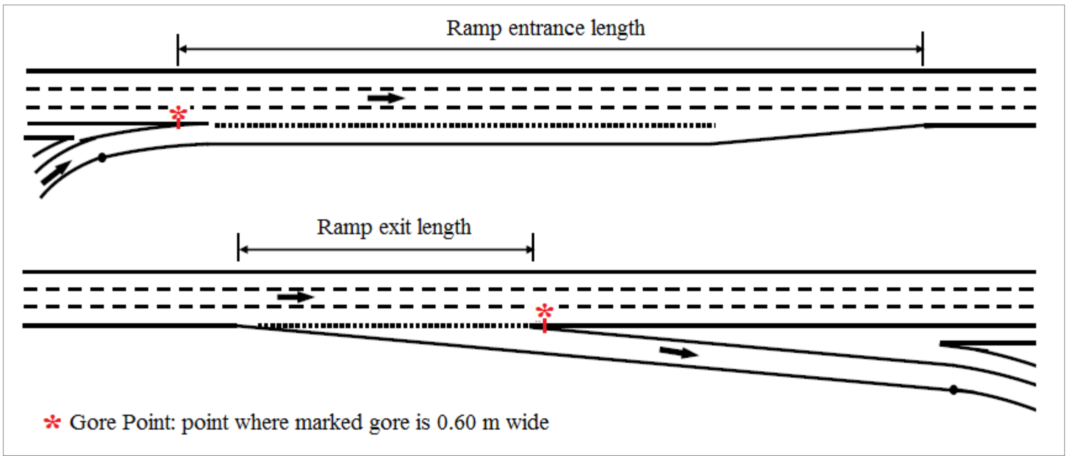



Figure 3.25: Gore point for entrance and exit ramps

**Outside shoulder width**

The CMFs describing the relationship between the average outside shoulder width and the predicted crash frequency have been computed by using the following equation:

$$CMF_{8,sv} = \left(1-\sum_{i=1}^{m} P_{c,i}\right)\cdot \exp[-0.0647\cdot(W_s-10)] + \left(\sum_{i=1}^{m} P_{c,i}\right)\cdot \exp[-0.0897\cdot(W_s-10)] \tag{3.14}$$

Where:

- $CMF_{8,y}$ is the crash modification factor for the average outside shoulder width of a freeway segment for fatal + injury single-vehicle crashes;
- $W_s$ is the paved outside shoulder width (ft);
- $P_{c,i}$ is the ratio of the length of curve $i$ within the segment to the length of the freeway segment.

The CMFs values for straight segments ($P_{c,i}$ = 0) as a function of the paved outside shoulder widths are plotted Figure 3.26. The CMF is applicable to shoulder widths ranging from 1.22 m to 4.27 m.

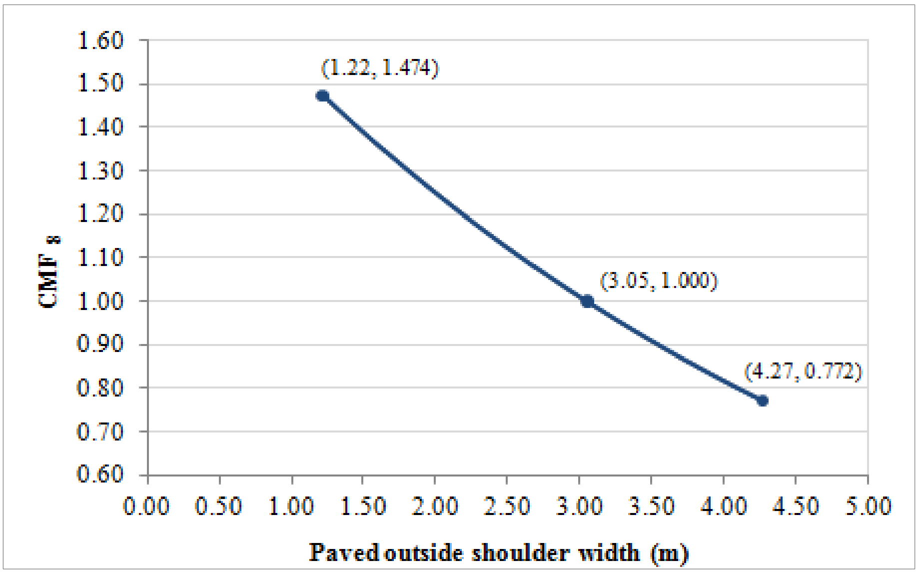



Figure 3.26: Relationship between average outside shoulder width and CMF value

**Outside barrier**

The CMFs describing the relationship between the roadside barrier presence in freeway segments and the predicted crash frequency have been computed by using the following equation:

$$CMF_{9,sv} = (1 - P_{ob}) + P_{ob} \cdot \exp\left(\frac{0.131}{W_{ocb}}\right) \tag{3.15}$$

Where:

- $CMF_{9,sv}$ is the crash modification factor for roadside barrier presence in a freeway segment for fatal + injury single-vehicle crashes;
- $P_{ob}$ is the proportion of segment length with a barrier present on the roadside;
- $W_{ocb}$ is the distance from edge of the paved outside shoulder to barrier face (ft).

Figure 3.27 shows the CMFs values as a function of the unpaved outside shoulder width ($W_{ocb}$) and for different values of the proportion of segment length with a roadside barrier ($P_{ob}$).

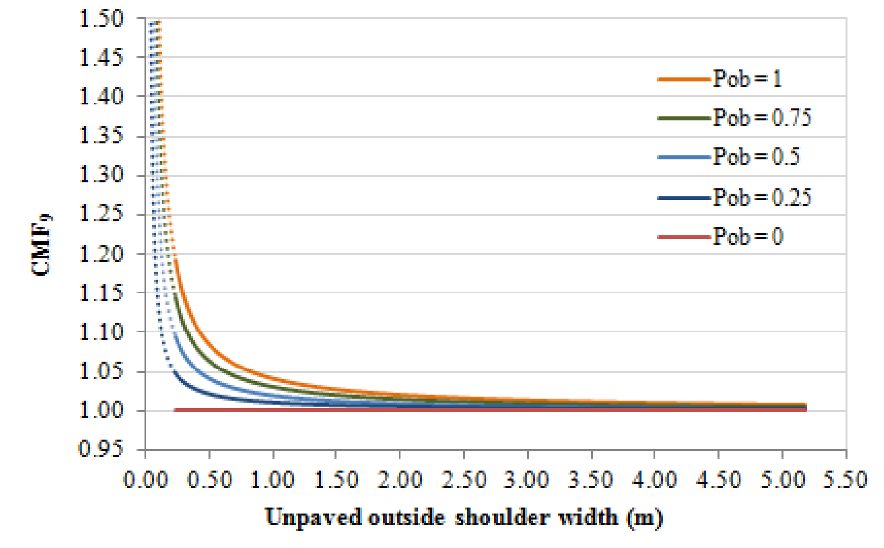



Figure 3.27: Relationship between unpaved outside shoulder width and CMF value

The HSM recommends values of $W_{ocb}$ in the range of 0.23 m to 5.18 m as the applicability domain of the CMF. The distance from the edge of the outside shoulder to barrier face is typically equal to zero in Italian motorways. Similarly to what observed for the unpaved inside shoulder, the CMF strongly increases for $W_{ocb}$ values lower than 0.23 m. The minimum value of 0.23 m was therefore used for the calculations.

As a consequence of this assumption, the CMFs values for fatal + injury single-vehicle crashes are those plotted in Figure 3.28 as a function of the proportion of segment length with roadside barrier.

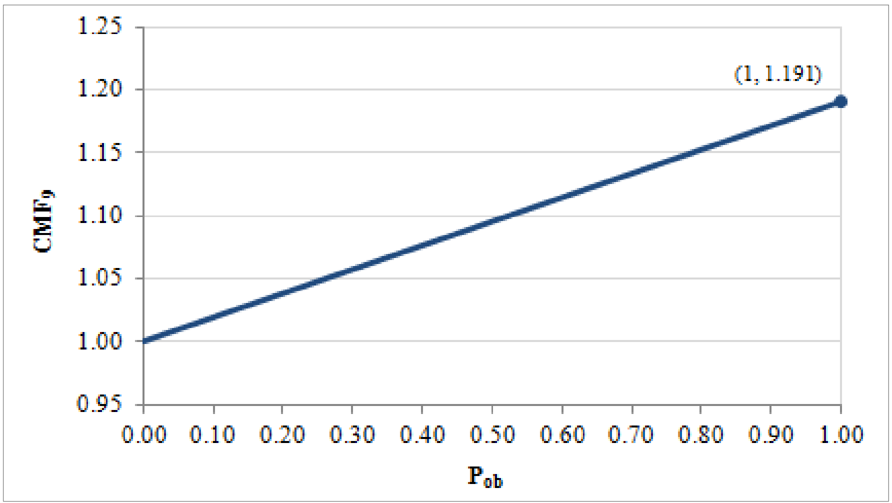



Figure 3.28: Relationship between $P_{ob}$ and CMF value

**Outside clearance**

The CMFs describing the relationship between the average outside clearance in freeway segments and the predicted crash frequency have been computed by using the following equation:

$$CMF_{10,sv} = \left(1 - P_{ob}\right) \cdot \exp[-0.00451 \cdot (W_{hc} - W_s - 20)] + 1.0907 \cdot P_{ob} \qquad (3.16)$$

Where:

- $CMF_{10,sv}$ is the crash modification factor for outside clearance in a freeway segment for fatal + injury single-vehicle crashes;
- $W_{hc}$ is the clear zone width (ft).

The CMF is applicable to clear zone widths less than 9.1 m and to shoulder widths in the range from 1.22 m to 4.27 m.

### *3.3.5 Calibration procedure*

The calibration of the safety performance functions is needed as crash frequencies of roadway are known to vary widely among countries and regions due to differences in climate, animal population, driver populations, and accident reporting thresholds and practices (Tarko, 2006). Each country should therefore develop and input its own calibration factor in order to adjust model estimates to be more comparable to the crash experience. The different models can be calibrated to each year of the study period by using calibration factors that also reflect time trends related to the crash frequency and the traffic volume.

The calibration procedure described in the Appendix B of the HSM (AASHTO, 2014) has been used to estimate the annual calibration factors (C).

This procedure consists of five different steps:

1. Identification of the prediction models to be calibrated;
2. Selection of sites for the calibration sample;
3. Data collection for each set of calibration sites for the calibration period;
4. Application of the prediction model to estimate the predicted average crash frequency by severity for each site during the calibration period;
5. Estimation of the annual calibration factors.

**Identification of the prediction models to be calibrated**

Calibration is performed separately for each predictive model. HSM identifies the prediction models to be calibrated as a function of the cross section and type crash.

According to HSM, the calibrations of prediction models for fatal + injury multi-vehicle and single-vehicle crashes are required for the specific study.

**Selection of sites for the calibration sample**

A reference group of sites was extracted from the motorway segment database and used to estimate the annual calibration factors to be included in the prediction model.

The reference group consisted of “untreated” sites (sites where roadworks were not carried out) with similar characteristics to the “treatment sites” (sites where roadworks occurred). The similarity was determined on the basis of the geometric features (e.g. three-lane or four-lane median divided motorways in rural or urban areas) and similar AADT ranges. Sites of the reference group are located at least 2 km upstream of the “road work” sign, to limit the effects of possible traffic queues, and 0.5 km downstream from the “end of road work“ sign.

HSM recommends a minimum sample size for the reference group between 30 to 50 sites with a length between 0.16 km and 1.60 km and recommends at least 100 observed crashes per year for the prediction model to be calibrated.

The sample of “untreated “sites did not meet these recommendations as a very large amount of the analyzed motorway segments were occupied by at least one work zone during the 6-year period from 2007 to 2012.

In order to obtain a larger sample size, sites where roadworks lasted less than one 24 hours (and more than 12 hours) were also added to the reference group sample. Of these, only work zones in which no crashes had occurred were selected for the reference group. The effect of these very short work activities (typically lane closures) on the annual crash frequency is small enough to allow to consider these sections as “untreated”. As a result, the calibration sample consisted of 920 motorway segments. The calibration sample characteristics are shown in Table 3.13.

**Data collection for each set of calibration sites**

The calibration database included information about all fatal + injury crashes occurred within each site during the calibration period consistent with the prediction model to be calibrated. Site characteristics data were needed to apply the prediction model for the same calibration period.

Table 3.13: Calibration sample characteristics

| Variable | Value |
|---|---|
| ***Total number of segments*** | ***920*** |
| ***Length of segments*** | |
| *Total* | *914 km* |
| *Average* | *0.99 km* |
| *Minimum* | *0.50 km* |
| *Maximum* | *1.10 km* |
| ***Number of fatal + injury crashes (2007-2012)*** | |
| *Total* | *5877* |
| *Average* | *6.39* |
| *Minimum* | *0* |
| *Maximum* | *44* |

**Application of the prediction model**

The prediction model is applied without using the EB method and without employing any other calibration factor. Indeed, the EB correction cannot be applied without an SPF calibrated for local conditions. In this study existing SPFs (from the HSM) have been used as predictive models in place of developing jurisdiction-specific models. As a consequence the existing SPFs should be previously calibrated for local conditions in order to apply the EB correction for the estimation of the expected average crash frequency.

Model calibration is performed by applying a multiplicative factor to the given SPF so that the aggregate number of predicted crashes is equal to the aggregate number of observed crashes throughout a jurisdiction.

Through this process, the predicted average crash frequency for each crash type is obtained for each site in the set of calibration sites and for each year in the calibration period.

**Estimation of the annual calibration factors**

The SPFs contain only traffic volume and road geometric variables and are estimated by using combined data during the study period. Thus, they are not able to account for the annual fluctuation in crash frequency caused by confounding factors, such as weather conditions, driver habits, enforcement levels, vehicle technologies or safety systems. Such causal factors may vary substantially over time and cause changes in crash counts unrelated to the treatment. The different models should therefore be calibrated to each year of the study period by using calibration factors that reflect time trends related to crash frequency and traffic volume.

The calibration factors were estimated for each year of analysis by using the following equation:

$$C_{y,x} = \frac{\sum_x N_{OBS,y,x}}{\sum_x N_{PRED,y,x}} \tag{3.17}$$

Where:

- $C_{y,x}$ is the calibration factor to adjust SPF of a freeway segment with crash type *y* (single-vehicle, multi-vehicle) in the year x;
- $N_{OBS,y,x}$ is the observed number of fatal + injury crashes in a freeway segment of the reference group with crash type *y* (single-vehicle, multi-vehicle) in the year x (crashes/yr);
- $N_{PRED,y,x}$ is the predicted average fatal + injury crash frequency of a freeway segment of the reference group with crash type *y* (single-vehicle, multi-vehicle) in the year x (crashes/yr).

The values of C for each year of analysis (2007-2012) are shown in Table 3.14 and plotted in Figure 3.29.

Table 3.14: Calibration factors for the years 2007 to 2012

| Year | Observed Crashes (multi-vehicle) | Predicted Crashes (multi-vehicle) | Observed Crashes (single-vehicle) | Predicted Crashes (single-vehicle) | Calibration Factor (multi-vehicle) | Calibration Factor (single-vehicle) |
|---|---|---|---|---|---|---|
| 2007 | 871 | 486 | 338 | 983 | 1.79 | 0.34 |
| 2008 | 717 | 510 | 302 | 1,000 | 1.41 | 0.30 |
| 2009 | 698 | 516 | 314 | 1,006 | 1.35 | 0.31 |
| 2010 | 658 | 514 | 270 | 1,001 | 1.28 | 0.27 |
| 2011 | 669 | 505 | 262 | 989 | 1.33 | 0.26 |
| 2012 | 516 | 457 | 262 | 944 | 1.13 | 0.28 |

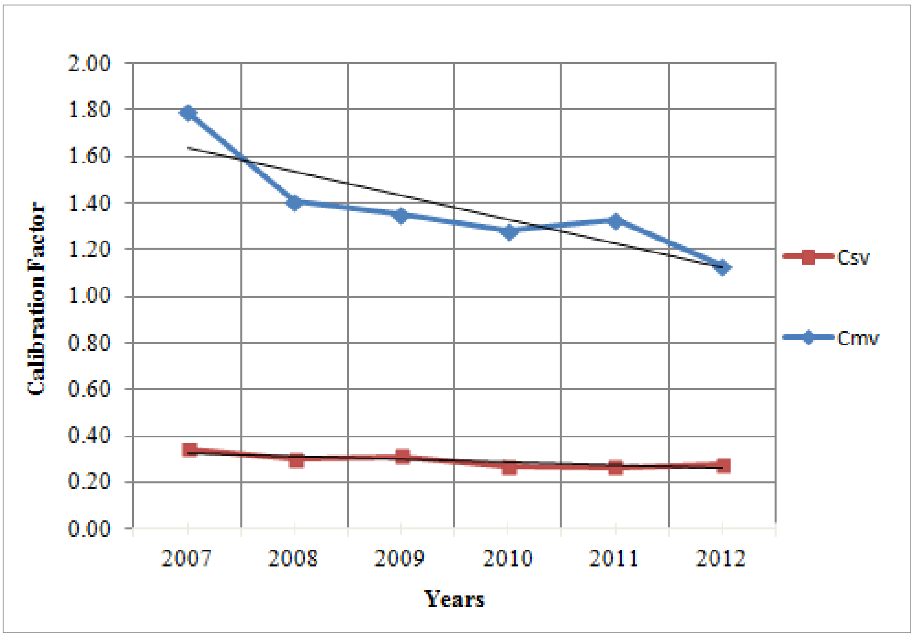



Figure 3.29: Annual calibration factors for the period 2007-2012

The results indicate a decreasing trend of severe crashes over the 6 years of analysis, especially for multi-vehicle crashes due to some external factors in addition to changes in the geometric features or functional characteristics of segments whose effects over the years are already included in the prediction models. This increased safety can be attributed to a number of factors such as driver education or communication campaigns promoted over the years or to the implementation of innovative speed management methods and traffic control devices in combination with advances in vehicle safety technology. Furthermore, the introduction of new regulations which introduce increased fines or demerit points for aggressive drivers may have encouraged a more careful driving behaviour.

On the other hand, the decreasing trend of crashes could be also related to the Regression-to-the-mean (RTM) effect that is the natural tendency of observed crashes to regress to the mean in the year following an unusually high or low crash count.

Specific evaluation techniques (e.g. EB method) are required in order to extract the RTM effect from the overall safety improvement recorded over the years.

The estimated calibration factors show that HSM models for freeway segments tend to overestimate fatal + injury single-vehicle crashes (C<1) and to underestimate fatal + injury multiple vehicle crashes (C>1).

These differences could be due to several potential reasons such as difference in climate, driver behaviour and accident reporting practices between Italy and the jurisdictions from which the HSM freeway models were developed. The HSM models for freeways were built by using freeway segment data of California, Maine and Washington. Differences in AADT values, driver behaviour and climate may be major causes of such a different distribution of crash types as compared to the Italian case. Furthermore, different crash reporting procedures and thresholds may introduce a significant difference in observed crash proportions.

### *3.3.6 Estimation of the work zone CMFs*

The objective of the Empirical Bayes (EB) methodology is to estimate the number of crashes that would have occurred at an individual treated site in the after period without treatment taking also care of the random fluctuations of crashes.

The intent of this procedure is to estimate $N_{EXP,A}$, which is the expected number of crashes that would have occurred in the after period without the work zone and compare that with $N_{OBS,A}$, the number of recorded crashes during the work zone activities over the same time period of interest.

The equations used in the EB procedure can be found in Gross et al. (2010). The EB estimate of the expected number of crashes in the before period at each work zone site, $N_{EXP,B}$, is computed as the weighted sum of the actual crashes in the before period and predicted crashes.

$$N_{EXP,B} = N_{PRED,B} \cdot w + N_{OBS,B} \cdot (1-w) \tag{3.18}$$

Where:

- $N_{PRED,B}$ is the predicted number of crashes in the before period;
- $N_{OBS,B}$ is the observed number of crashes in the before period;
- $w$ is the prediction model weight.

The weighted adjustment factor $w$ is computed as follows:

$$w = \frac{1}{(1 + k \cdot \sum N_{PRED,B})} \tag{3.19}$$

The prediction model weight is derived by using the over-dispersion parameter (k) given for the specific model used by the equation (3.5).

The expected number of crashes that would have occurred in the after period without work zone ($N_{EXP,A}$) is estimated as follows:

$$N_{EXP,A} = N_{EXP,B} \cdot \frac{N_{PRED,A}}{N_{PRED,B}} \tag{3.20}$$

Where:

- $N_{EXP,B}$ is the unadjusted empirical Bayes estimate;
- $N_{PRED,B}$ is the predicted number of crashes estimated for the before period;
- $N_{PRED,A}$ is the predicted number of crashes estimated for the after period.

The variance of $N_{EXP,A}$ is estimated from $N_{EXP,A}$, the before and after predictions and the EB weight as:

$$Var\ (N_{EXP\ ,A}) = N_{EXP\ ,A} \cdot \frac{N_{PRED\ ,A}}{N_{PRED\ ,B}} \cdot (1-w) \tag{3.21}$$

The estimates of $N_{EXP,A}$ calculated for each work zone site are then summed over all work zone sites with a given layout configuration and compared with the crashes counts during the after period ($N_{OBS,A}$). The variance of $N_{EXP,A}$ is also summed over all sites with that configuration.

The index of safety effectiveness of the analyzed condition ($\theta_s$ or $CMF_{w,s}$) for a given layout configuration "s" is estimated as follows:

$$\theta_s = \frac{\sum_{i=1}^{ns} N_{i,OBS,A} / \sum_{i=1}^{ns} N_{i,EXP,A}}{1 + \sum_{i=1}^{ns} Var(N_{i,EXP,A}) / (\sum_{i=1}^{ns} N_{i,EXP,A})^2} = CMF_{w,s} \tag{3.22}$$

Where:

- $ns$ is the number of work zones with a given layout configuration "s".

The standard deviation of $\theta_s$ is given by

$$Std.dev(\theta_s) = [Variance(\theta_s)]^{0.5} \tag{3.23}$$

With:

$$Variance(\theta_s) = \frac{\theta_s^2 \cdot [1/\sum_{i=1}^{ns} N_{i,OBS,A} + \sum_{i=1}^{ns} Var(N_{i,EXP,A}) / (\sum_{i=1}^{ns} N_{i,EXP,A})^2]}{[1 + \sum_{i=1}^{ns} Var(N_{i,EXP,A}) / (\sum_{i=1}^{ns} N_{i,EXP,A})^2]^2} \tag{3.24}$$

For each work zone at least one year of observations were used for the before period. On the other hand, work zone duration averaged at 5 days with the shortest being 12 hours.

As a consequence, before and after periods may have very different durations and the pre-work zone periods are in some cases several months longer than the during-work zone periods. It is likely that the crash frequency during the roadworks would be lower than that in the before period simply because few crashes occurred within these work zones due to their short duration. In such circumstances, the estimation of the crash frequency reduction during the work zone period compared to the pre-treatment period could be biased.

The relative duration of the before and after periods is therefore a key variable that impacts the sample size requirements and the level of confidence in the observed changes would increase as the after period lengthens. However, by increasing the minimum duration threshold the sample size drastically decreases.

## 3.4 Results and discussion

Table 3.15 shows the CMF, the standard deviation of the CMF, the number of crashes that occurred during the work zone and the expected number of crashes in the after period for each work zone configuration. Furthermore for each CMF a 95% confidence interval is shown. The CMF is determined to be statistically significant if the specified confidence interval of the CMF does not include 1.0, since a value of 1.0 indicates no effect from the intervention.

Table 3.15: Results of the Empirical-Bayes analysis

| **Layout (two-lane carriageway)** | **$\Sigma N_{OBS}$** | **$\Sigma N_{EXP}$** | **CMF** | **Std. Dev. (CMF)** | **95% C.I. (lower limit)** | **95% C. I. (upper limit)** |
|---|---|---|---|---|---|---|
| Cross2(1+1) | 8 | 2.52 | **3.11** | 0.56 | 2.01 | 4.22 |
| Cross2(0+1) | 56 | 26.82 | **2.08** | 0.09 | 1.90 | 2.27 |
| Fast2(2) | 60 | 36.44 | **1.64** | 0.08 | 1.49 | 1.80 |
| Slow2 | 20 | 12.33 | **1.62** | 0.12 | 1.39 | 1.85 |
| Emergency2 | 73 | 57.32 | **1.27** | 0.04 | 1.20 | 1.35 |
| Fast2 | 21 | 19.50 | 1.08 | 0.06 | 0.95 | 1.20 |
| **Layout (three-lane carriageway)** | **$\Sigma N_{OBS}$** | **$\Sigma N_{EXP}$** | **CMF** | **Std. Dev. (CMF)** | **95% C.I. (lower limit)** | **95% C. I. (upper limit)** |
| Cross3(1+1) | 9 | 3.14 | **2.80** | 0.52 | 1.79 | 3.81 |
| Cross3(0+1) | 5 | 2.28 | **2.15** | 0.52 | 1.13 | 3.16 |
| Slow&Middle3 | 4 | 2.03 | 1.91 | 0.56 | 0.81 | 3.01 |
| Middle&Fast3 | 3 | 1.52 | 1.90 | 0.71 | 0.52 | 3.29 |
| Fast3(3) | 14 | 9.17 | **1.51** | 0.18 | 1.16 | 1.87 |
| Fast3 | 23 | 15.37 | **1.49** | 0.10 | 1.29 | 1.70 |
| Cross3(0+2) | 16 | 12.81 | **1.25** | 0.10 | 1.05 | 1.44 |
| Slow3 | 31 | 30.09 | 1.03 | 0.05 | 0.93 | 1.13 |
| Emergency3 | 77 | 77.29 | 1.00 | 0.04 | 0.92 | 1.07 |
| Middle&Fast3(2) | 14 | 16.58 | **0.84** | 0.07 | 0.70 | 0.99 |
| **All** | **434** | **325.23** | **1.33** | **0.02** | **1.30** | **1.37** |

The CMFs resulted statistically significant at the 95 percent confidence level are shown in bold in Table 3.15. The values of CMFs together with their 95% confidence intervals are also plotted in Figure 3.30.

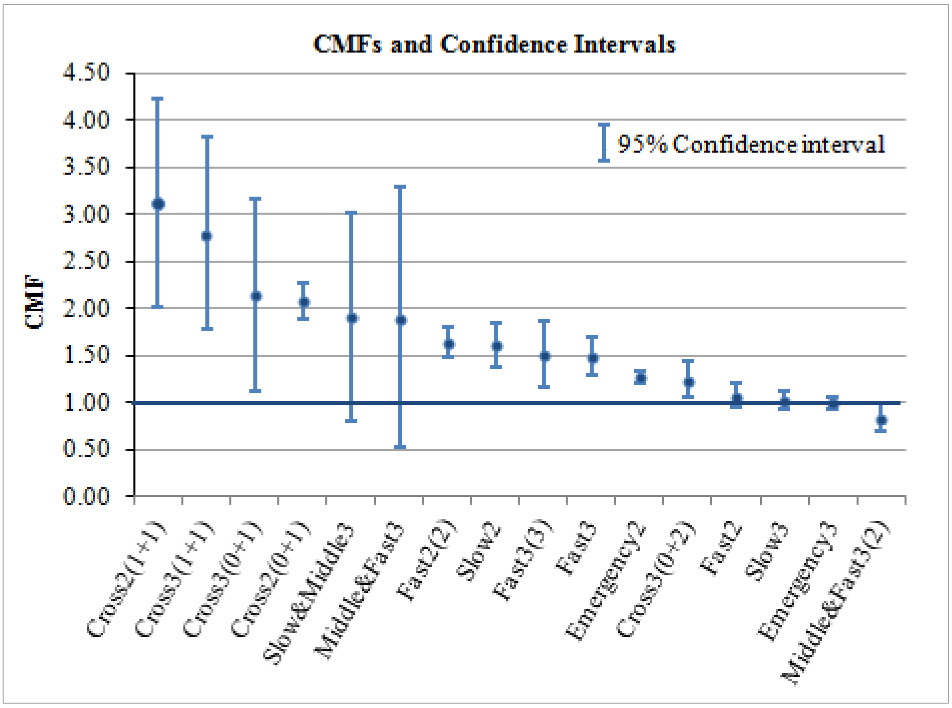



Figure 3.30: Crash Modification Factors for each configuration

Overall, the EB procedure estimated a significant 33% increase in the expected crash frequency due to the installation of work zones.

The results show that most of layouts that involve a crossover, such as the layouts “Cross2(1+1)”, “Cross3(1+1)”, “Cross3(0+1)” and “Cross2(0+1)” are associated with CMF values greater than 2. The highest value of CMF (3.11) is observed for the work zone configuration with partial diversion of traffic to the opposite carriageway through a single-lane crossover with part of the traffic remaining in the ordinary flow direction (layout “Cross2(1+1)”, Figure 3.31) and the second highest value (2.80) is associated to the layout where traffic is partially diverted to the slow lane and to the opposite carriageway through a single-lane crossover (layout “Cross3(1+1)”, Figure 3.31).

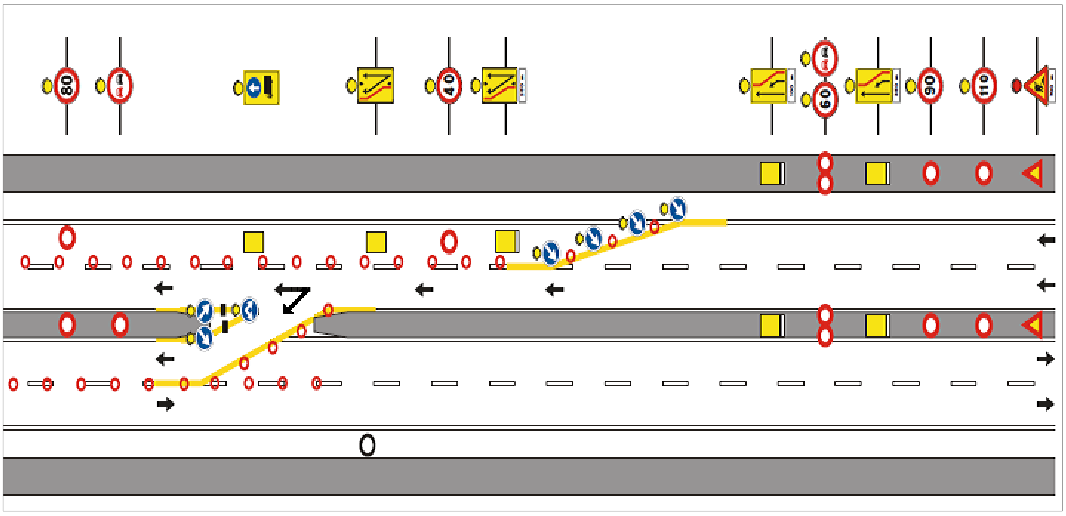


Figure 3.31: Partial diversion of traffic to the overtaking lane and to the opposite carriageway through a single-lane crossover

This finding seems to indicate that this type of configurations, which requires the drivers to choose whether to travel on the normal carriageway or move to the opposite one, represent a critical issue for user safety. Driver's uncertainty during lane change manoeuvres in correspondence of the median opening is probably a major cause of such a high expected crash frequency.

The configuration with dual-lane crossover (layout "Cross3(0+2)") is associated with a much lower CMF value (1.25) as compared to those estimated for the other crossover configurations. This result indicates the number of lanes diverted to the opposite carriageway as a key variable to identify an optimal crossover design.

All work zone configurations seem to lead to an increase in severe crash frequencies with the exception of the layout "Middle&Fast3(2)" associated with a CMF value equal to 0.84. The installation of this particular work zone configuration seems to provide safer conditions for users resulting in a 16% reduction in the expected crash frequency. This could be related to the speed reduction that affects the road safety more than the actual physical impairment.

Finally, the CMFs for the layouts "Slow&Middle3", "Middle&Fast3", "Fast2", "Slow3", "Emergency3" that include 1.0 in their confidence intervals, should be considered as not significant at the 95% confidence level. Therefore these values should be used with caution.

# Chapter 4.

# Testing of countermeasures in virtual reality

### 4.1 Introduction

The purpose of the driving simulator experiments was to determine the safest and most effective countermeasures for the reduction of speed and speed variance within stationary work zones.

The results of the accident analysis have shown that work zones with median crossover are associated with the highest crash frequencies as compared to other types of work zones. A number of countermeasures to moderate speed behaviour have been therefore tested through a typical crossover layout, designed in accordance with the Italian Ministerial Decree 10 July 2002 (Ministero delle Infrastrutture e dei Trasporti, 2002).

Nine different crossover configurations have been designed and tested in the driving simulator of the Road Safety and Accident Reconstruction Laboratory (LaSIS) of the University of Florence (Italy). Five of the nine configurations have been designed and analyzed as a contribution to the ASAP project (Cocu et al., 2014)[3]. The driving simulation experiments, performed during the ASAP project, were focused on the analysis of speed variances in addition to that of mean actuated speeds. The experiments investigated the effects of different speed limit sequences and alternative design features, such as wider lanes and median openings.

The remaining four configurations have been designed at a later stage and added to the driving simulation study to further implement the ASAP research findings. A different approach, based on Human Factor (HF) principles for safer roads (PIARC,

[3] The results of such experiments are provided in the deliverable 4.1 of the ASAP Project - “Speed management at Work Zone - Field studies and stakeholder’s survey” (Cocu et al., 2014). The ASAP report provides the analysis of speed behaviour, performed on a partial sample composed of twenty-six subjects.

2008), has been tested in such configurations. This approach consisted in manipulating the visual environment by means of different traffic calming measures to unconsciously induce motorists to moderate their speed. The considered approach is conceptually similar to those that use pavement markings, such as chevrons or transverse bars whose effectiveness in reducing speeds has already been ascertained (Godley et al., 1999; Katz, 2007; Voigt and Kuchangi, 2008).

## 4.2 Material and methods

### *4.2.1 LaSIS Driving Simulator*

The LaSIS driving simulator (University of Florence, 2013) used for the tests is a medium-high fidelity dynamic simulator, equipped with a full scale vehicle fitted on a 6 degrees of freedom Stewart's platform, allowing roll, yaw and pitch (Figure 4.1).

Figure 4.1: LaSIS driving simulator at the University of Florence (Italy)

The driver, inside the cabin, is immersed in a virtual environment in which all the sensorial stimuli typical of driving are faithfully reproduced. The visual reproduction of the road scenario is obtained by means of four projectors installed on the ceiling, projecting on a cylindrical screen embracing an angle wider than 200 degrees. The three rear mirrors are replaced by 6.5" LCD monitors, reproducing the rear vision. The sound is generated by a multichannel audio system, capable to reproduce both the vehicle and the environmental noise. All the functions are supervised by a network of 5 computers, including an operator's station from which the simulation is managed.

### *4.2.2 Participants*

Forty-three subjects were recruited on a voluntary basis among students, staff of the University of Florence (Italy) and other volunteers from outside the University. The subjects were selected according to the following criteria:

- no previous experience with driving simulators;
- possession of a valid Italian driver's license;
- at least five years of driving experience;
- an annual driven distance greater than 5,000 km;
- low susceptibility to motion sickness.

One subject did not complete the experiment due to symptoms of simulator sickness and was therefore excluded from the analysis. Forty-two subjects (9 women and 33 men), aged between 24 and 50 years old (mean value: 36.1 years; standard deviation: 8.2 years) participated in the research. Their driving experience (measured in terms of years of driving license possession) varied between 5 years and 32 years (mean value: 16.8 years; standard deviation: 7.9 years).

### *4.2.3 Scenarios' design*

The analyzed scenarios are based on a 2+2 lane motorway with an ordinary speed limit of 130 km/h. The cross section of the carriageway is equal to that of the main Italian highways and it is composed by two lanes, each 3.75 m wide, and a 3 m wide emergency lane with a roadside barrier and a median barrier. The median is 2.60 m wide.

Particular attention has been placed on temporary signs and barriers, all built using a three dimensional software and introduced in the scenario. The experiments were carried out during daylight conditions and using dry pavement conditions.

Nine different configurations of the crossover layout were designed on the same 7 km long motorway section and implemented in the driving simulator.

A typical crossover layout, designed in accordance with the Italian Ministerial Decree 10 July 2002 (Ministero delle Infrastrutture e dei Trasporti, 2002), was defined as a reference configuration for the study. The speed behaviour through this work zone configuration, called configuration "0", has been compared with that of eight alternative work zone layouts, named configurations "0_VMS", "1", "2", "3", "4", "5", "6" and "7".

The configurations “0”, “0_VMS”, “1”, “2” and “3” have been designed and successively analyzed within the ASAP project (Cocu et al., 2014). The measures implemented in these configurations have been focused on homogenizing the speed, by facilitating the crossover manoeuvre or by increasing temporary speed limits.

The configurations “4”, “5”, “6” and “7” have been focused on obtaining the same result (homogenization of the driving speed) by means of different perceptual countermeasures based on HF principles (PIARC, 2012).

Detailed characteristics of each configuration are described in the following sections.

**Configuration “0”**

This type of work zone is a crossover in which the traffic flowing northwards is diverted to the opposite carriageway, where two traffic streams travel in opposite directions (Figure 4.2).

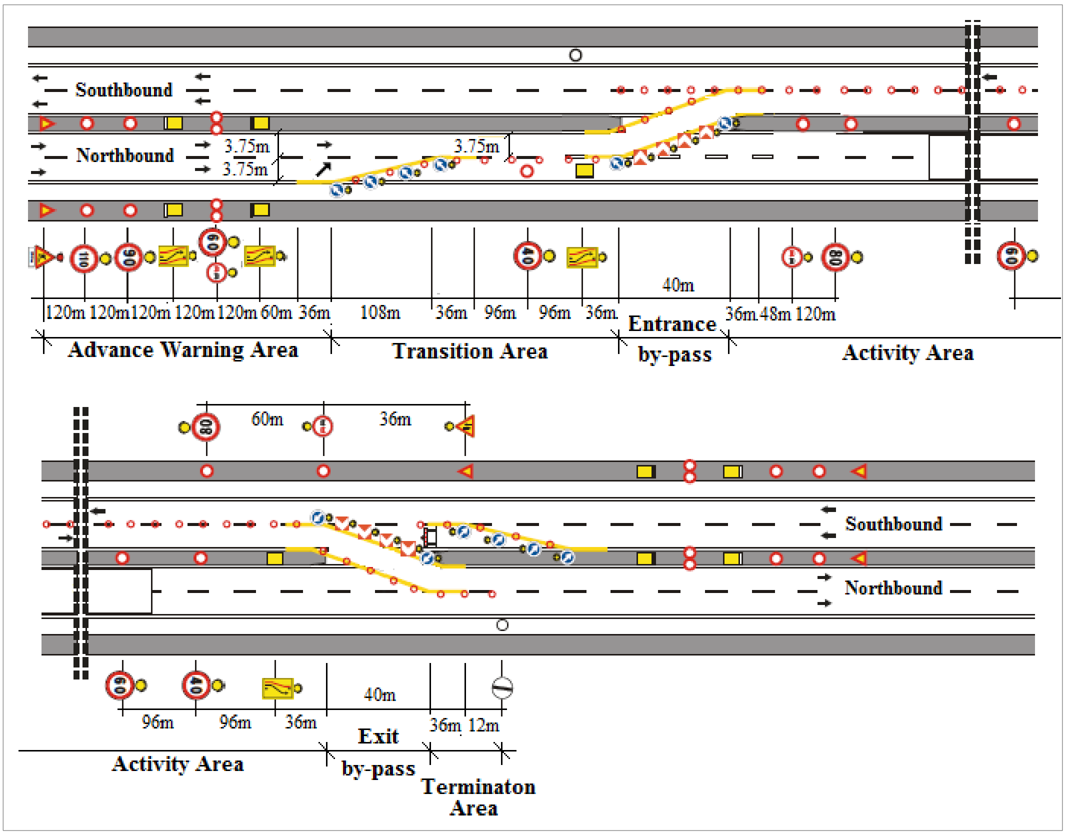



Figure 4.2: The crossover layout

The closure of the northbound carriageway occurs in two distinct stages:

1. closure of the slow lane with traffic diverted to the overtaking lane;

2. closure of the overtaking lane and total diversion of traffic to the opposite carriageway through a single-lane crossover.

The alignment implemented in the simulator is composed of the following sections:

- an initial 3,500 m long section of standard motorway layout;
- a work zone section of 3,380 m that includes the advance warning area (696 m), the transition area (372 m), the entrance by-pass (40 m wide), the activity area (2,184 m), the exit by-pass (40 m wide) and the termination area (48 m).

The signs are consistent with the Italian technical rules for temporary signs (Ministero delle Infrastrutture e dei Trasporti, 2002).

The advance warning area contains six pairs of signs with one sign located on each side of the roadway. The user encounters at first the “road work” signs (Figure 4.3, top left), then, the other traffic signs arranged at a distance of 120 m from one another. Specifically they consist of the 110 km/h speed limits, the 90 km/h speed limits, the “right lane closure” signs and the 60 km/h speed limit signs. The “right lane closure” signs are then the last pair of signs encountered by the user in the advance warning area.

Approximately 90 m after the “right lane closure” signs there is the transition area (Figure 4.3, top right), which consists of two distinct sections:

- a 108 m long merging taper (realized with delineators and “keep left” signs) that closes the slow lane and requires drivers to move on the overtaking lane;
- a 250 m long section on the overtaking lane where the speed limit is reduced to 40 km/h.

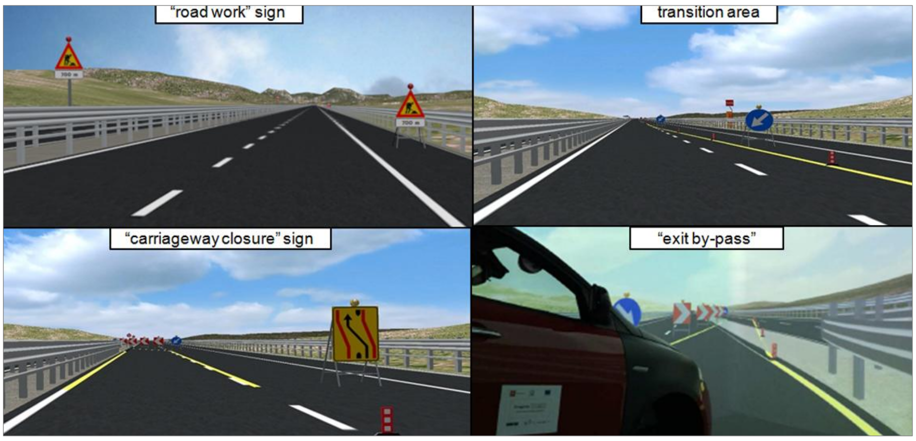



Figure 4.3: Work zone areas – reference configuration (configuration “0”)

The 40 km/h speed limit sign is placed about 100 m before the end of the transition area, followed by the “carriageway closure” sign placed 36 m before the entrance by-pass (Figure 4.3, bottom left) where traffic is diverted to the opposite carriageway through a single-lane crossover.

In correspondence of the activity area the opposite traffic flows are concentrated on the southbound carriageway, with a single lane for each travel direction.

The standard channelizing devices used to separate the traffic flows consist of 30 cm tall flexible delineators placed at a distance of 12 m from each other.

Moving along this section the user encounters a “No Overtaking” sign placed about 85 m after the by-pass and then, at a distance of 120 m, the 80 km/h speed limit that applies to all the activity area. The speed limit is subsequently reduced prior to 60 km/h at 228 m distance before the exit by-pass and then to 40 km/h before the 40 m wide median opening that moves the traffic back to their carriageway (Figure 4.3, bottom right).

The termination area includes the taper to direct the traffic back into the roadway after traversing the activity area. This area ends with the ”End of road work” sign, placed 48 m after the exit by-pass. The configuration “0” has been considered as the reference configuration.

**Configuration “0_VMS”**

This configuration is different from the “Configuration 0” due to the installation of a Variable Message Sign (VMS) in place of the “road work” sign on the right shoulder (Figure 4.4).


Figure 4.4: The VMS sign (configuration “0_VMS”)

The message implemented in the VMS sign reads “Riduci la velocità” (“Reduce the speed” in Italian). This choice was based on the findings of the literature review. The study performed by Garber and Patel (2001), that examined the effects of four different messages in Virginia, identified the message “You are speeding, slow down” as the most successful on reducing the mean speeds within work zones. This message successfully singled out drivers, as they perceived the meaning that this message was not a general warning.

**Configuration “1”**

The configuration “1” is different from the configuration “0” due to the wider median opening (80 m instead of 40 m for both the entrance and the exit by-pass). Furthermore the sequence of speed limits in the advance warning area is 110-80 km/h, instead of the sequence 110-90-60 km/h used in the configuration “0” (the 80 km/h limit in place of the 60 km/h limit and the 90 km/h speed limit sign is removed) and the 40 km/h speed limit in the by-pass is increased to 60 km/h (Figure 4.5). Also the speed limits within the activity area are different: the limit of 60 km/h and 40 km/h are increased respectively to 80 km/h and 60 km/h.

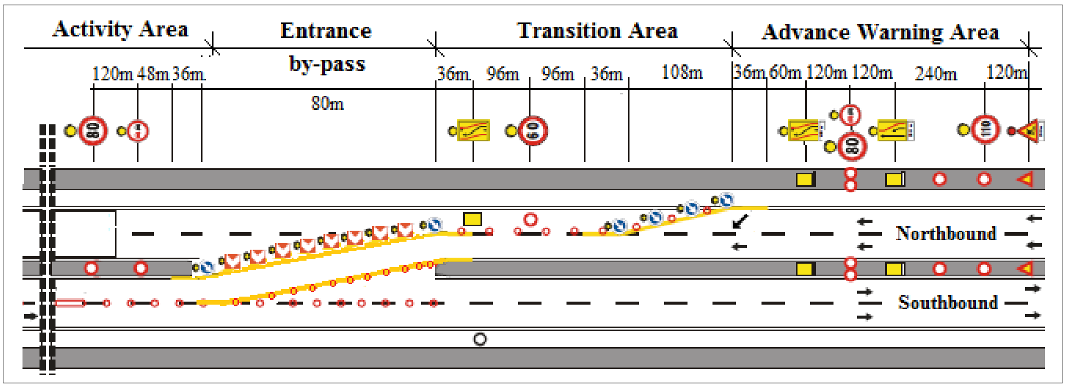



Figure 4.5: Work zone layout (configuration “1”)

**Configuration “2”**

The layout of traffic signs of configuration “2” is the same of that implemented in configuration “0”. Also the median opening is the same (40 m).

The lane width for traffic flow travelling through the work zone is increased from 3.75 m to 5 m. The 5 meter lane is achieved through the lateral displacement of delineators and yellow lines (with the original white lines left in place), as shown in Figure 4.6

Figure 4.6: Lateral displacement of delineators (configuration “2”)

**Configuration “3”**

Configuration “3” consists of the same sequence of signs and the same opening width as implemented in configuration “1”. The lane width, as in configuration “2”, is equal to 5m.

**Configuration “4”**

In configuration ”4”, a sequence of 1.10 m high vertical delineators, placed at a distance of 3 m from each other, replace the flexible delineators (0.30 m high) within the transition area (Figure 4.7).

Figure 4.7: Vertical delineators within the transition area (configuration “4”)

Larger sized chevron alignment signs, used to provide additional guidance for the crossover chicane, are also implemented in this configuration. Their dimensions decrease in the direction of the activity area: four chevron signs having dimensions

150x150 cm, 120x120 cm, 90x90 cm, 90x90 cm replace the 90x90 cm standard sized chevrons adopted in the reference configuration (Figure 4.8).

Figure 4.8: Chevron signs alignment

**Configuration "5"**

Configuration "5" has the same work zone layout as configuration "4", with the exception of having a 3 m tall panel placed in proximity of the entrance by-pass. This panel is made up of a series of black and yellow vertical stripes which increase in width in the travelling direction (Figure 4.9).

Figure 4.9: Panel with visual patterns (configuration "5")

There are, two different visual patterns on the panel: the first one (Figure 4.10, upper part) is 36 m long and runs, from section G to section H, parallel to the travel direction. The second one (Figure 4.10, lower part) is 40.25 m long and, starting from section H, runs parallel to the chevron sign alignment..

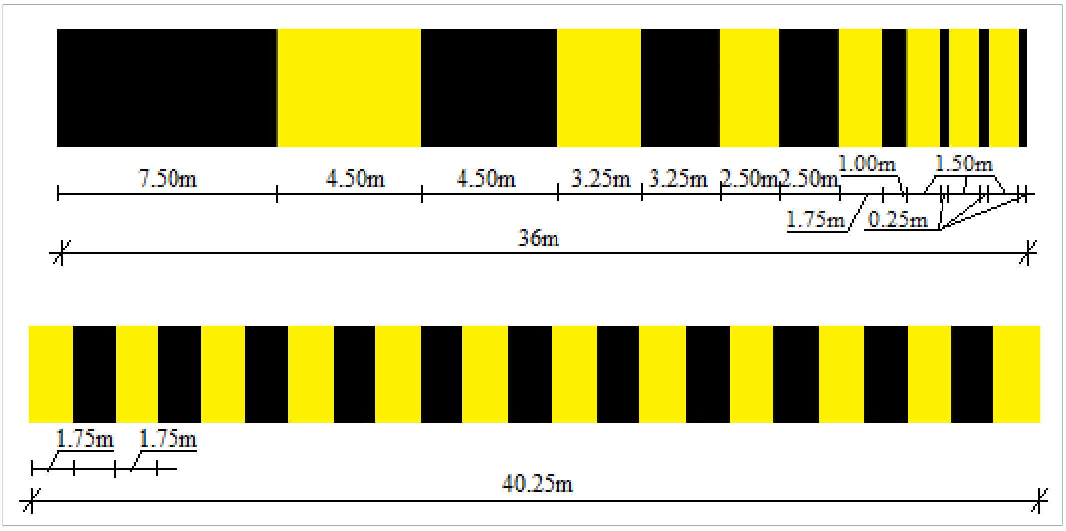



Figure 4.10: Visual pattern conditions

**Configuration "6"**

In configuration "6", the 1.10 m high vertical delineators are placed also within the advance warning area (located on the white line of the outside shoulder, starting from site B) and not only within the transition area. The larger sized chevron signs are also installed in this configuration. Furthermore, all speed limit signs are removed (Figure 4.11).

Figure 4.11: Transition area (configuration "6")

**Configuration "7"**

In configuration "7", the sequence of vertical delineators and the larger size chevron signs are designed as in configuration "6". Furthermore, a visual pattern consisting of alternating red and white stripes is painted on the median barrier starting from site B (Figure 4.12). Each strip is 1 m wide.


Figure 4.12: Visual pattern on the median barrier (configuration "7")

### *4.2.4 Testing procedure*

Upon arrival at the laboratory, each participant was briefed on the experimental design, and asked to read and sign an informed consent form. The participants were given some basic information about the use of the simulator, warned about simulator sickness, and informed that they could stop the test at any time.

They were then asked to wear the safety belt and drive as they normally would, although they were not briefed about the research objectives.

Drivers then performed a 10-minute training phase in order to familiarize with the vehicle and its control instruments, such as the steering wheel, gearbox, accelerator and brakes. The training scenario consisted of a motorway section with moderate traffic.

At the end of the training phase, the subject was asked to get down from the cabin and fill in a post-training questionnaire (Appendix C). The participant then took a 5-minutes break before starting the experimental session.

In order to reduce data collection bias, each participant encountered each of the nine configurations in varying random order.

### *4.2.5 Data collection and statistical analysis*

Although the simulator collected a great number of parameters, the study focused on the analysis of speed and deceleration based on the findings of the literature review that identified speeding and especially speed variance as major causes in work zone crashes.

The comparison between speeds, collected at a sampling rate of 20 Hz, was carried out in the following sections (Figure 4.13):

- at an upstream section located 500 m before the "work zone" sign (site A);

- at the “road work” sign (site B);
- at each speed limit sign (sites C, D, E, F, I, J, K);
- at the “carriageway closure” sign (site G);
- at the beginning section of the entrance by-pass (site H) and of the exit by-pass (site L).

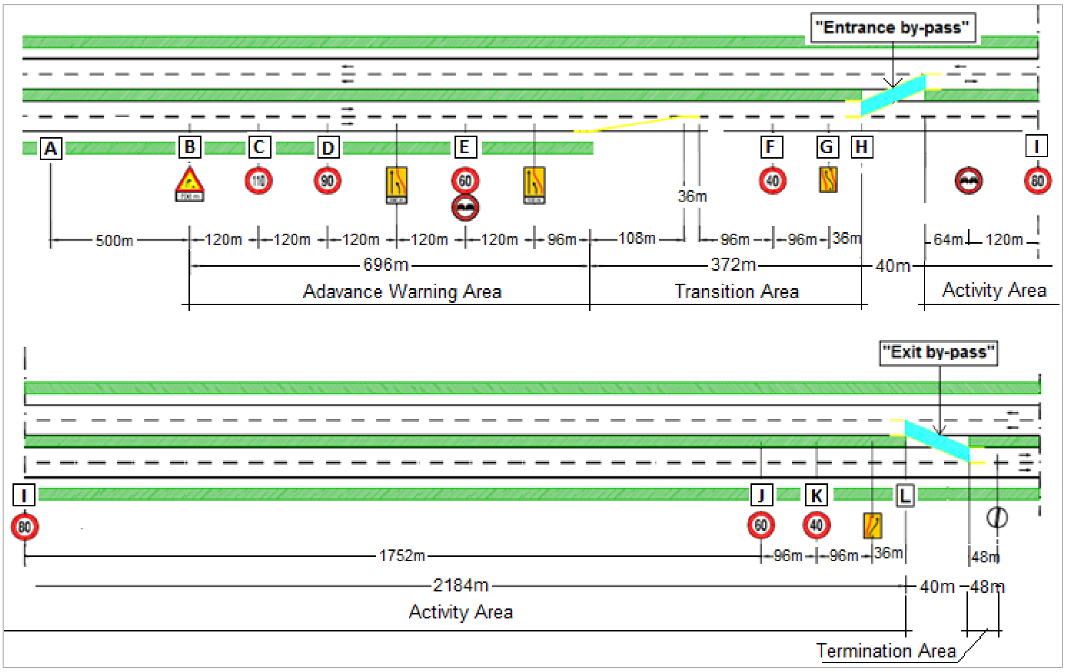



Figure 4.13: Speed measurement sites (configuration “0”)

A comparative analysis of speeds between the reference configuration and the alternative configurations has been conducted in order to determine if differences between mean speeds at each site were significant. For the comparative analysis, a bilateral t-test for paired samples has been performed at a level of significance of 5%.

The purpose of the paired t-test is to determine whether there is statistical evidence that the mean difference between paired observations is significantly different from zero.

Parametric tests such as the paired t-test assume that the data are normally distributed. The Shapiro-Wilk normality test (Shapiro and Wilk, 1965) was therefore conducted at each work zone site to make sure that differences between pairs were normally distributed.

In order to determine the statistical significance of mean speeds differences, a hypothesis was postulated and then the validity of that hypothesis was tested. In this study, the following two hypothesis were evaluated:

- $H_0$ (null hypothesis): the actual speeds in a specific site of two different work zone configurations belong to the same population (the mean speeds are equal);
- $H_a$ (alternative hypothesis): the two samples do not belong to the same population (the mean speeds are not equal).

The test statistic is a t-score ($t$) defined by the following equation:

$$t = \frac{\bar{d}}{\sqrt{s_d^{\,2} / n}} \tag{4.1}$$

where:

- $\bar{d}$ is the difference between the means of the two samples;
- $s_d^2$ is the sample variance;
- n is the sample size.

The standard deviation ($s_d$) of the differences computed from $n$ matched pairs is estimated as:

$$s_d = \sqrt{\frac{\sum d_i - \bar{d}}{n-1}} \tag{4.2}$$

where $d_i$ is the difference for pair $i$.

The p-value associated with the t-score is calculated by using the distribution of the test statistic with n-1 degree of freedom and then compared to the significance level. In this specific case, the null hypothesis is rejected when the p-value is less than 0.05.

Furthermore, in order to evaluate the deceleration behaviour in approaching the advance warning and transition areas, the driver's reactions on the accelerator and on the brake pedals have been detected (Figure 4.14). Mean deceleration rates have been computed by identifying sudden changes in the accelerator or brake pedal positions.

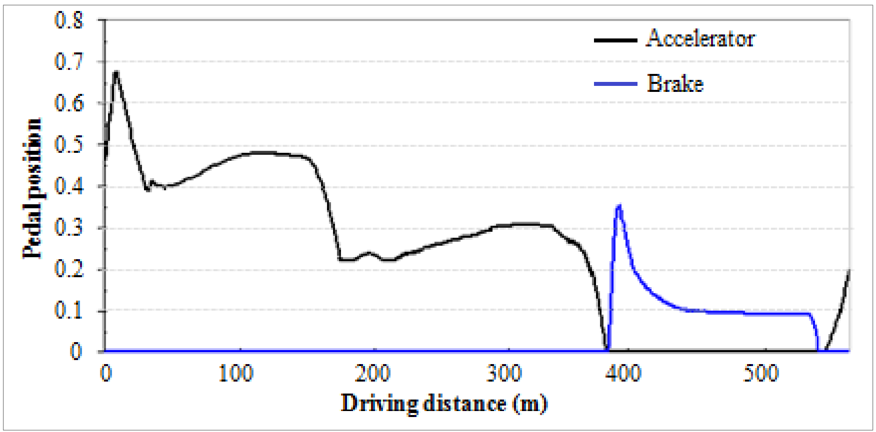



Figure 4.14: Example of accelerator and brake pedals input

## 4.3 Results and discussion

For the comparative analysis between configurations, the mean speeds, standard deviations and speed variances were calculated. The values of mean speed, standard deviation and variance calculated for each measurement site are shown in Table 4.1 and Table 4.2.

Table 4.1: Summary of results (from site A to site F)

| **Configuration** | **Variable** | **Measurement site** | | | | | |
|---|---|---|---|---|---|---|---|
| | | **A** | **B** | **C** | **D** | **E** | **F** |
| 0 | Mean Speed (km/h) | 129.03 | 123.12 | 117.32 | 111.74 | 103.81 | 74.14 |
| | Std. Dev. (km/h) | 9.59 | 12.14 | 13.23 | 13.26 | 13.22 | 14.98 |
| | Variance (km²/h²) | 92.03 | 147.48 | 175.09 | 175.87 | 174.67 | 224.39 |
| 0_VMS | Mean Speed (km/h) | 128.23 | 119.08 | 112.39 | 108.41 | 102.04 | 75.57 |
| | Std. Dev. (km/h) | 9.51 | 12.48 | 10.62 | 11.67 | 12.79 | 14.97 |
| | Variance (km²/h²) | 90.43 | 155.75 | 112.68 | 136.22 | 163.48 | 224.10 |
| 1 | Mean Speed (km/h) | 128.55 | 120.53 | 116.23 | 113.53 | 104.46 | 74.08 |
| | Std. Dev. (km/h) | 9.04 | 12.61 | 13.55 | 13.12 | 13.62 | 11.91 |
| | Variance (km²/h²) | 81.70 | 158.91 | 183.62 | 172.05 | 185.64 | 141.80 |
| 2 | Mean Speed (km/h) | 129.85 | 125.85 | 119.32 | 112.77 | 102.97 | 76.66 |
| | Std. Dev. (km/h) | 9.36 | 12.96 | 13.32 | 13.90 | 14.63 | 14.38 |
| | Variance (km²/h²) | 87.59 | 167.91 | 177.30 | 193.29 | 214.02 | 206.72 |
| 3 | Mean Speed (km/h) | 128.64 | 124.41 | 119.21 | 114.07 | 106.55 | 78.10 |
| | Std. Dev. (km/h) | 9.93 | 11.66 | 13.62 | 14.73 | 14.44 | 14.94 |
| | Variance (km²/h²) | 98.62 | 136.07 | 185.59 | 217.08 | 208.57 | 223.08 |
| 4 | Mean Speed (km/h) | 128.35 | 126.28 | 120.32 | 113.19 | 103.23 | 72.01 |
| | Std. Dev. (km/h) | 9.77 | 13.48 | 14.86 | 13.68 | 14.02 | 12.52 |
| | Variance (km²/h²) | 95.48 | 181.63 | 220.85 | 187.11 | 196.62 | 156.76 |
| 5 | Mean Speed (km/h) | 128.76 | 122.16 | 116.11 | 110.21 | 101.30 | 69.52 |
| | Std. Dev. (km/h) | 9.70 | 13.25 | 14.37 | 14.69 | 14.07 | 12.93 |
| | Variance (km²/h²) | 94.14 | 175.58 | 206.44 | 215.87 | 198.05 | 167.28 |
| 6 | Mean Speed (km/h) | 128.47 | 119.85 | 117.24 | 114.44 | 107.12 | 77.00 |
| | Std. Dev. (km/h) | 9.21 | 13.27 | 14.36 | 14.82 | 14.87 | 13.56 |
| | Variance (km²/h²) | 84.86 | 176.06 | 206.14 | 219.76 | 221.17 | 183.94 |
| 7 | Mean Speed (km/h) | 128.61 | 117.36 | 110.22 | 106.04 | 97.29 | 69.08 |
| | Std. Dev. (km/h) | 9.83 | 14.53 | 14.97 | 15.61 | 13.97 | 11.99 |
| | Variance (km²/h²) | 96.57 | 211.19 | 224.03 | 243.63 | 195.13 | 143.76 |

Table 4.2: Summary of results (from site G to site L)

| **Configuration** | **Variable** | **Measurement site** | | | | | |
|---|---|---|---|---|---|---|---|
| | | **G** | **H** | **I** | **J** | **K** | **L** |
| 0 | Mean Speed (km/h) | 54.59 | 50.42 | 82.52 | 95.34 | 82.32 | 53.50 |
| | Std. Dev. (km/h) | 11.68 | 10.02 | 7.20 | 12.89 | 12.65 | 9.95 |
| | Variance (km²/h²) | 136.38 | 100.45 | 51.82 | 166.03 | 159.95 | 99.01 |
| 0_VMS | Mean Speed (km/h) | 53.56 | 49.69 | 82.82 | 95.90 | 82.23 | 51.40 |
| | Std. Dev. (km/h) | 11.40 | 11.09 | 7.32 | 13.24 | 13.49 | 9.24 |
| | Variance (km²/h²) | 130.03 | 122.89 | 53.59 | 175.37 | 182.09 | 85.45 |
| 1 | Mean Speed (km/h) | 64.80 | 63.35 | 87.82 | 95.63 | 83.28 | 70.18 |
| | Std. Dev. (km/h) | 9.44 | 9.24 | 8.50 | 12.83 | 13.30 | 10.39 |
| | Variance (km²/h²) | 89.13 | 85.30 | 72.23 | 164.72 | 176.92 | 107.94 |
| 2 | Mean Speed (km/h) | 53.09 | 52.96 | 87.59 | 99.59 | 85.39 | 56.31 |
| | Std. Dev. (km/h) | 11.21 | 11.93 | 9.11 | 14.87 | 13.20 | 10.27 |
| | Variance (km²/h²) | 125.61 | 142.23 | 82.96 | 221.19 | 174.33 | 105.47 |
| 3 | Mean Speed (km/h) | 67.27 | 67.09 | 92.39 | 99.16 | 85.74 | 73.14 |
| | Std. Dev. (km/h) | 11.86 | 12.18 | 9.99 | 15.40 | 14.75 | 13.55 |
| | Variance (km²/h²) | 140.60 | 148.28 | 99.72 | 237.05 | 217.48 | 183.48 |
| 4 | Mean Speed (km/h) | 56.43 | 51.37 | 81.26 | 93.70 | 78.63 | 53.44 |
| | Std. Dev. (km/h) | 10.52 | 9.19 | 8.94 | 15.81 | 15.35 | 9.13 |
| | Variance (km²/h²) | 110.71 | 84.44 | 79.95 | 249.95 | 235.69 | 83.34 |
| 5 | Mean Speed (km/h) | 57.85 | 51.12 | 82.38 | 92.31 | 79.15 | 54.90 |
| | Std. Dev. (km/h) | 9.57 | 9.67 | 8.73 | 15.62 | 16.32 | 11.04 |
| | Variance (km²/h²) | 91.52 | 93.50 | 76.14 | 243.85 | 266.32 | 121.78 |
| 6 | Mean Speed (km/h) | 55.53 | 49.66 | 81.82 | 90.11 | 77.92 | 54.81 |
| | Std. Dev. (km/h) | 11.28 | 11.46 | 9.57 | 15.13 | 15.30 | 9.06 |
| | Variance (km²/h²) | 127.13 | 131.31 | 91.61 | 229.05 | 234.10 | 82.15 |
| 7 | Mean Speed (km/h) | 55.21 | 52.08 | 82.70 | 91.44 | 78.45 | 54.10 |
| | Std. Dev. (km/h) | 9.27 | 9.30 | 11.36 | 15.82 | 15.94 | 9.62 |
| | Variance (km²/h²) | 85.98 | 86.53 | 129.01 | 250.28 | 254.10 | 92.55 |

Considering the theory that the safest work zones are those with the smallest increase in the upstream-to-work-zone speed variance (Migletz et al., 1998), the changes in speed variance between an upstream section and a section inside of the work zone, have also been used as a safety indicator in this study.

Speed variances have been calculated in an upstream section (site A) and in a section inside (site G) of the work zone (Figure 4.15). Successively the percentage change between them has been calculated.

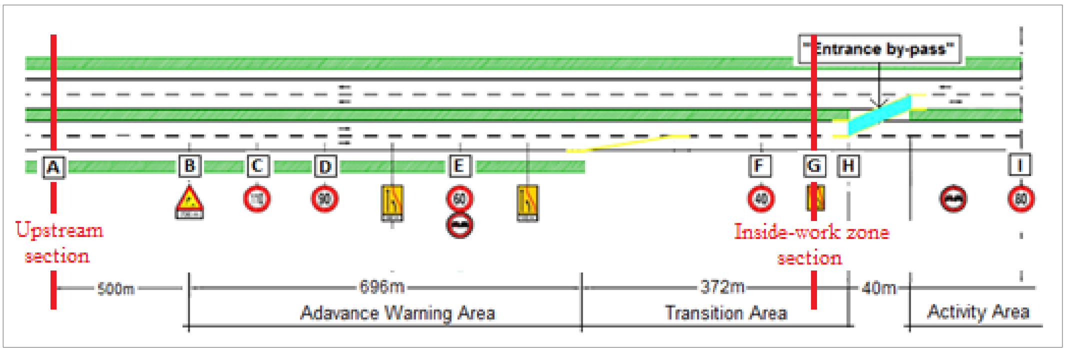



Figure 4.15: Upstream and work zone sections for speed variance analysis

Site A, located 500 m upstream from the “road work” sign, has been chosen as the upstream section in the speed variance analysis. This choice is motivated by the fact that the drivers, because of the great distance, could not see the work zone from this section, and their speed behaviour is therefore not affected by the work zone presence.

Site G is located within the transition area, between the last speed limit sign and the entrance by-pass, and has been chosen as the inside-work zone section in the speed variance analysis. This site is located downstream from the work zone areas, within which the countermeasures are implemented and the changes in speed limit sequence or lane width occur.

The speed variances computed for each configuration are shown in Figure 4.16 and the percentage changes between site A and site G are shown Figure 4.17.

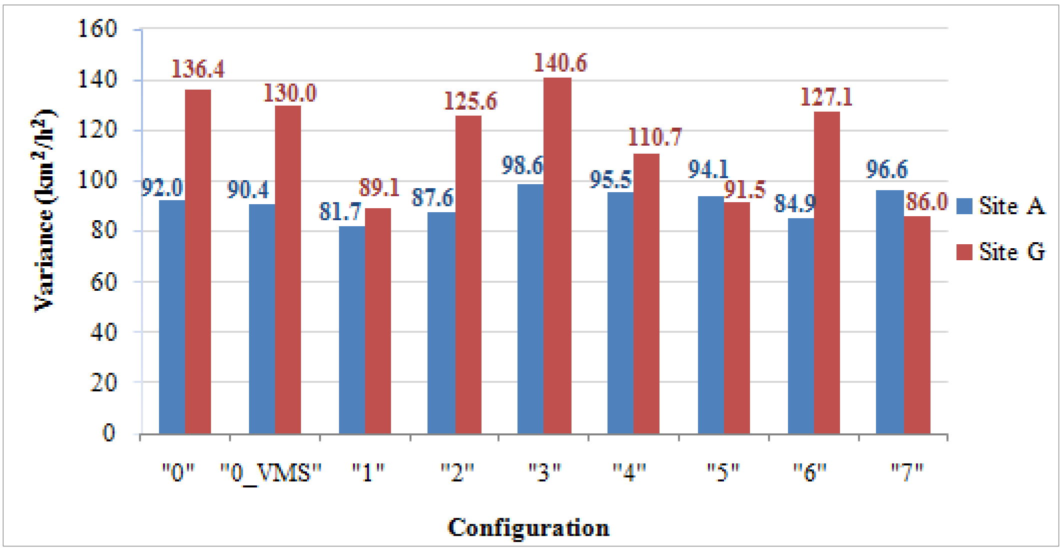



Figure 4.16: Comparison between speed variances at site A and site G

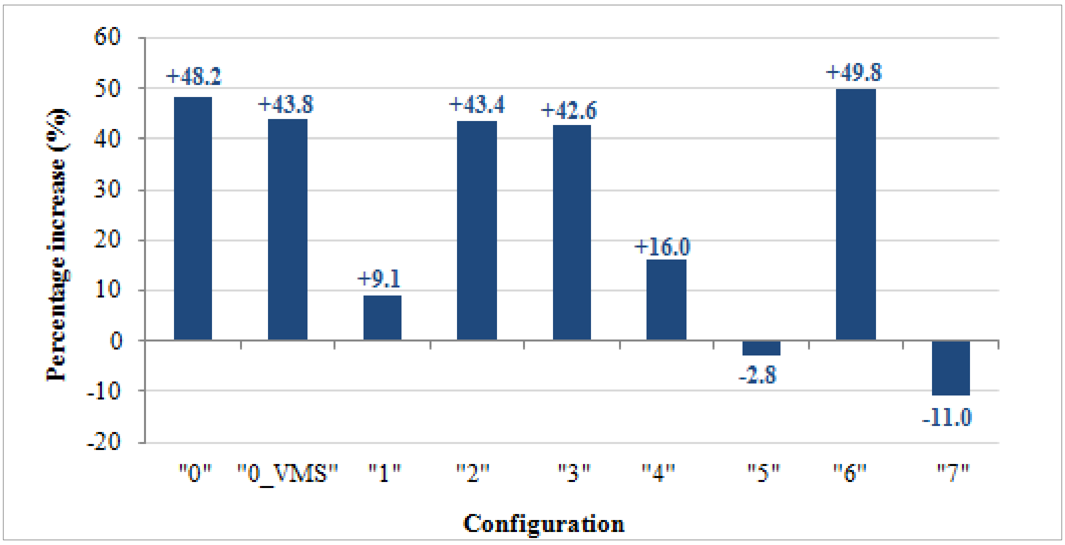



Figure 4.17: Percentage change in speed variance from site A to site G

### *4.3.1 Speed behaviour in the reference configuration*

The speed analysis performed on the total sample of 42 participants in configuration "0", showed a speed behaviour perfectly in line with that of the ASAP project, which used a partial sample of 26 participants (Cocu et al., 2014).

In this configuration the average speed recorded upstream of the work zone (site A) is about 129 km/h (Figure 4.18), with the normal speed limit being of 130 km/h.

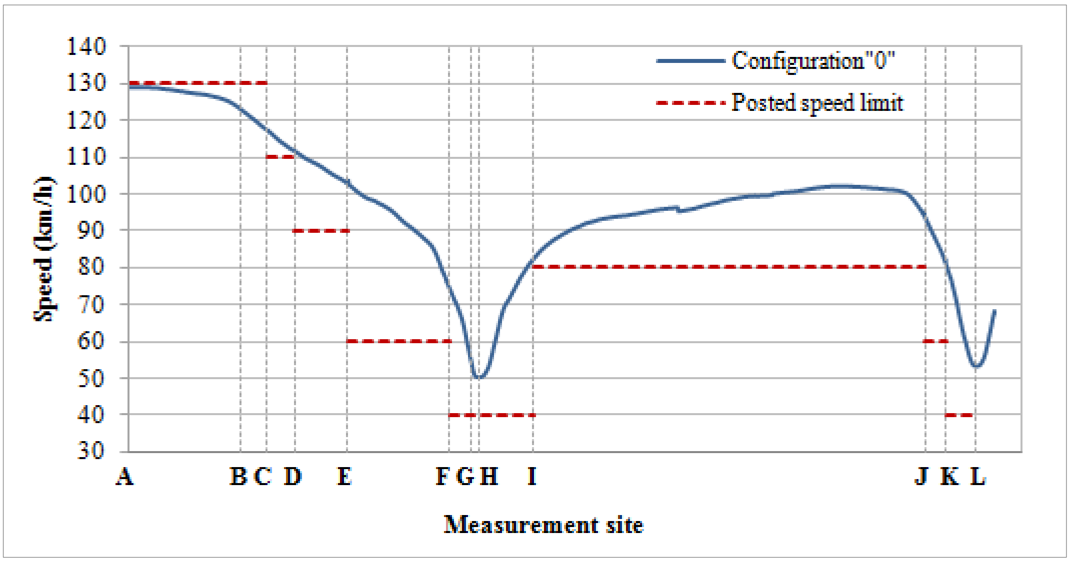



Figure 4.18: Mean speed profile of configuration "0"

Drivers approach the warning area with a mean speed of 123 km/h at the "road work" sign (site B) and start to progressively reduce their speed.

At the 110 km/h speed limit sign (site C) and in proximity of the 90 km/h limit sign (site D) the mean speeds are respectively 117 km/h and 112 km/h. Drivers then adopt a mean speed of 100 km/h at site E, despite the 60 km/h posted limit and a mean speed still 35 km/h higher than the temporary limit at site F.

Two distinct phases of deceleration can be identified from the analysis of mean speed profile before the entrance by-pass. The users start to slow down in proximity of site B with a mean deceleration of 0.35 $m/s^2$, then, starting from a section located approximately 250 m upstream of the entrance by-pass, they significantly reduce their speed a higher deceleration rate equal to 0.81 $m/s^2$.

Even in approach of the 40 m wide entrance by-pass (site H), where the flow is diverted to the opposite carriageway through a single-lane crossover, the mean speed is about 50 km/h, still higher than the posted speed limit of 40 km/h.

The mean speed recorded in the activity area is always higher than the posted limit: the users start accelerating after exiting the by-pass and reach a maximum speed value higher than 100 km/h. At a distance of about 250 m (similar to the driving behaviour in approaching the entrance by-pass) the drivers perceive the presence of the exit by-pass and reduce their speeds with a mean deceleration of about 1.14 $m/s^2$. The mean speed at the exit by-pass (site L) is 53.5 km/h.

These results show that the mean speeds within each work zone area are always higher than those prescribed by the temporary speed limits and decrease significantly only when the drivers recognize the presence of the by-passes perceiving them as a hazard.

The percentage increase in speed variance between upstream (site A) and inside the work zone (site G) was about 48 %.

### *4.3.2 The effect of the Variable Message Sign*

In configuration “0_VMS” a Variable Message Sign (VMS) was installed in place of the “road work” static sign (site B), in order to investigate its effectiveness on speed reductions.

The mean speed recorded at the upstream section (site A) is approximately the same for both configurations. This is likely due to the fact that drivers don’t yet perceive the presence of the work zone at this distance.

The mean speed measured in correspondence of the VMS (site B) is 119 km/h, 4 km/h lower than that recorded at the "work zone" static sign of the reference configuration. This difference is maintained in the following section (site C) where the speed in the configuration "0_VMS" is 112 km/h. Afterwards, the benefit of the VMS decreases and disappears at site E.

### *4.3.3 The impact of changes in speed limits*

The configuration "1" was designed to verify the effect of a different speed limit sequence within the advance warning and the transition areas (110-80-60 km/h, in place of the reference sequence 110-90-60-40 km/h). The comparison between the mean speeds profiles of configuration "0" and configuration "1" are shown in Figure 4.19.

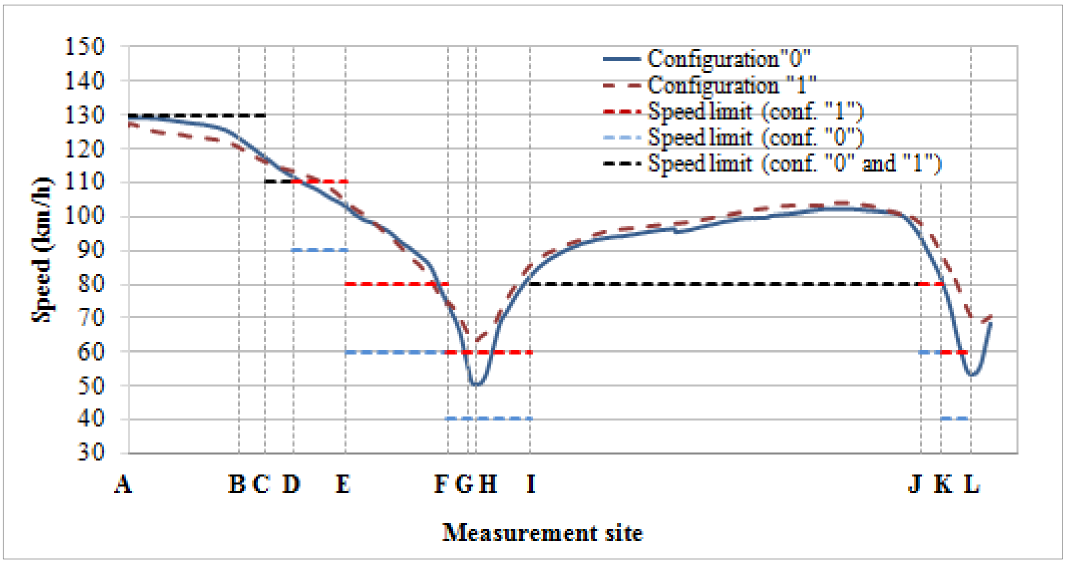



Figure 4.19: Comparison between mean speed profiles (configurations "1" and "0")

Although the speed limit sequence is consistently changed, the mean speed profile does not change significantly and only at the entrance and exit by-passes a significant increase of speeds is observed, likely due to the wider median opening (80 m instead of 40 m). As a consequence, a greater compliance with speed limits occurs in the configuration "1 (Table 4.3 and Table 4.4).

Figure 4.20 shows the speeding behaviour (defined as the difference between the mean speed and the posted speed limit) at each measurement site of configurations "0" and "1".

Table 4.3: Speeding behaviour in configurations “0” and “1” (from site A to site F)

| Configuration | Variable | Measurement site | | | | | |
|---|---|---|---|---|---|---|---|
| | | **A** | **B** | **C** | **D** | **E** | **F** |
| 0 | Speed limit (km/h) | 130 | 130 | 110 | 90 | 60 | 40 |
| | Mean speed (km/h) | 129.03 | 123.12 | 117.32 | 111.74 | 103.81 | 74.14 |
| | Speeding (km/h) | -0.97 | -6.88 | +7.32 | +21.74 | +43.81 | +34.14 |
| 1 | Speed limit (km/h) | 130 | 130 | 110 | 110 | 80 | 60 |
| | Mean speed (km/h) | 126.95 | 120.53 | 116.23 | 113.53 | 104.46 | 74.08 |
| | Speeding (km/h) | -3.05 | -9.47 | +6.23 | +3.53 | +24.46 | +14.08 |

Table 4.4: Speeding behaviour in configurations “0” and “1” (from site G to site L)

| Configuration | Variable | Measurement site | | | | | |
|---|---|---|---|---|---|---|---|
| | | **G** | **H** | **I** | **J** | **K** | **L** |
| 0 | Speed limit (km/h) | 40 | 40 | 80 | 60 | 40 | 40 |
| | Mean speed (km/h) | 54.59 | 50.42 | 82.52 | 95.34 | 82.32 | 53.5 |
| | Speeding (km/h) | +14.59 | +10.42 | +2.52 | +35.34 | +42.32 | +13.5 |
| 1 | Speed limit (km/h) | 60 | 60 | 80 | 80 | 60 | 60 |
| | Mean speed (km/h) | 64.8 | 63.35 | 87.82 | 95.63 | 83.28 | 70.18 |
| | Speeding (km/h) | +4.8 | +3.35 | +7.82 | +15.63 | +23.28 | +10.18 |

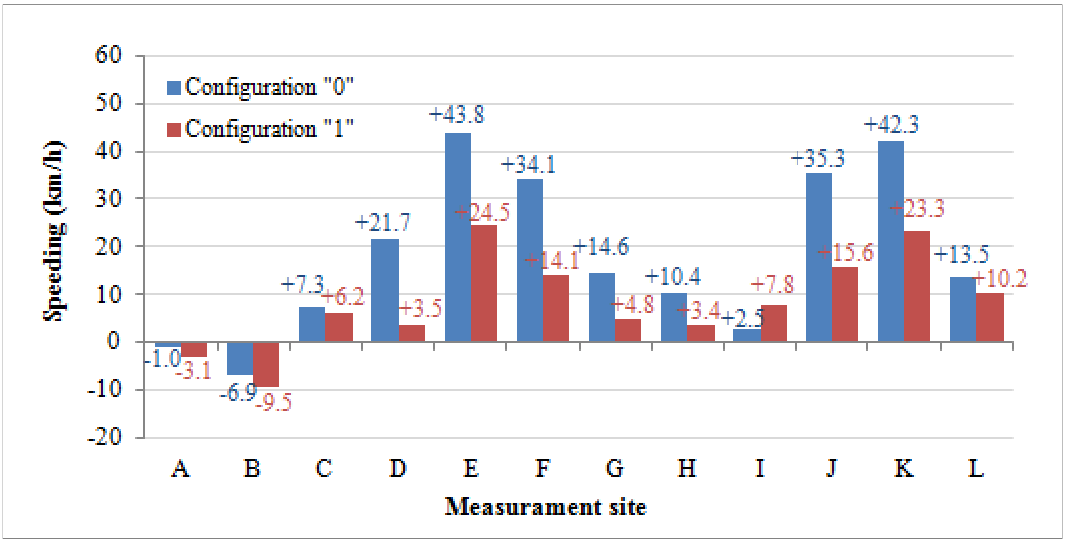



Figure 4.20: Speeding behaviour in configurations “0” and “1”

Furthermore, a smoother variation of the actuated deceleration can be observed when approaching the transition area: instead of actuating the deceleration in two distinct phases, as in configuration “0”, the deceleration gradually increases in three different phases from 0.25 $m/s^2$ to 0.42 $m/s^2$ and then to 0.51 $m/s^2$.

The reduced value of the final deceleration is very likely due to the widening of the median opening implemented in this configuration.

The analysis of the percentage change of speed variances from site A to site G of configuration “1” shows a smaller increase in speed variance (+9.1%), compared to that measured in the configuration “0” (+48.2%). Therefore, based on the literature experimental evidence that smaller changes in speed variance between upstream to inside work zone cause a lower potential for crashes, the sequence of speed limits (together with the 80 m opening width) implemented in configuration “1” seems to provide safer conditions for drivers.

The speed profile recorded in configuration “6”, where all speed limits are removed and perceptual countermeasures implemented, is similar to that of configurations “0” and “1” (Figure 4.21).

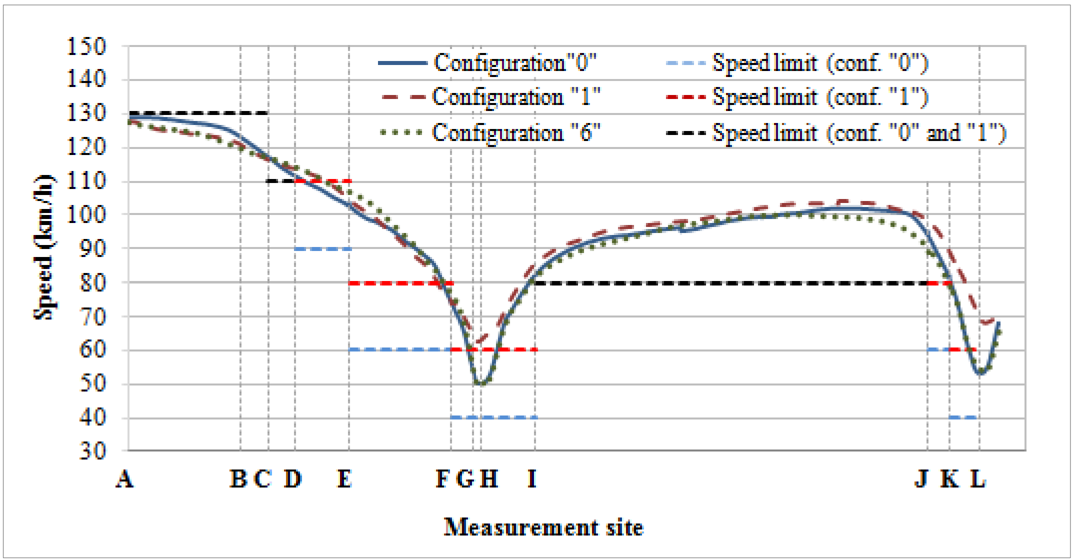



Figure 4.21: Comparison between mean speed profiles (configurations “0”, “1” and “6”)

This evidence demonstrates that the speed actuated by drivers is mainly influenced by the field of view rather than by the posted speed limits. No changes in the speed profile occur by increasing the speed limits without changing the optical density of the field of view and still the same speed profile is attained when the posted speed limits are removed and the optical density of the field of view is increased.

### *4.3.4 The effect of a wider median opening*

In order to verify the effect of increasing the median opening width in configuration "1" (80 m in place of 40 m), the mean values recorded in sites H and L were compared with those adopted in the reference configuration.

The comparison shows that the speed increases as the median opening increases. The mean speed increases from 50.4 km/h to 63.3 km/h at the entrance by-pass (site H), and from 53.5 km/h to 70.2 km/h at the exit bypass (site L) when the opening width is increased from 40 to 80 m. Such differences are statistically significant according to the t-test results (Table 4.5).

Furthermore, the mean deceleration recorded between the 40 km/h speed limit sign (site F) and the entrance by-pass (site H) is lower when the drivers approach the 80 m opening width (-0.51 $m/s^2$) as compared to that recorded in approaching the 40 m by-pass (-0.81 $m/s^2$ or more).

According to these results it can be concluded that a larger width of the median opening allows the users to complete the manoeuvre safely even at higher speeds, avoiding sudden decelerations or abrupt manoeuvres.

### *4.3.5 The effect of the increase in lane width*

In order to investigate the effect of the increased lane width (5 m instead of 3.75 m), the mean speeds along the transition and the activity areas of configuration "2" have been analyzed.

The mean speed profiles (Figure 4.22) show no significant changes within the advance warning and the transition areas, while in the activity area the speed values with the 5 m wide lane are always higher than those recorded with a 3.75 m lane.

The analysis of the change in speed variances shows a similar value (+43.4%) than the one recorded for the configuration "0" (+48.2%). According to this result, a wider lane width does not seem to provide safer conditions to drivers.

Furthermore, the mean speeds recorded within the entrance and exit by-passes of the configuration "2" are slightly higher than those recorded in the configuration "0". The lane width is therefore a factor that influences the speeds within the by-pass independently from its width.

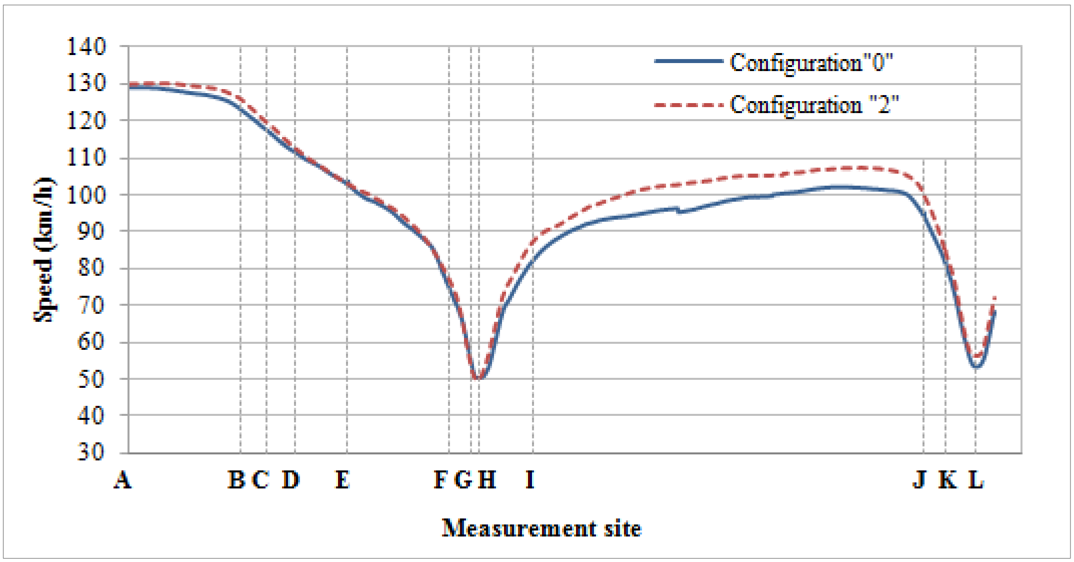



Figure 4.22: Comparison between mean speeds profiles (configurations “2” and “0”)

#### *4.3.6 The combined effect of a wider median opening and lane*

The analysis of the configuration “3” was performed in order to evaluate, with respect to configuration “0”, the effects of the implementation of the larger median opening (80m) together with the wider lane (5 m).

The speeds recorded in the configuration “3” are, as expected, higher than those recorded in the reference configuration, especially within the by-passes and within the activity area (Figure 4.23).

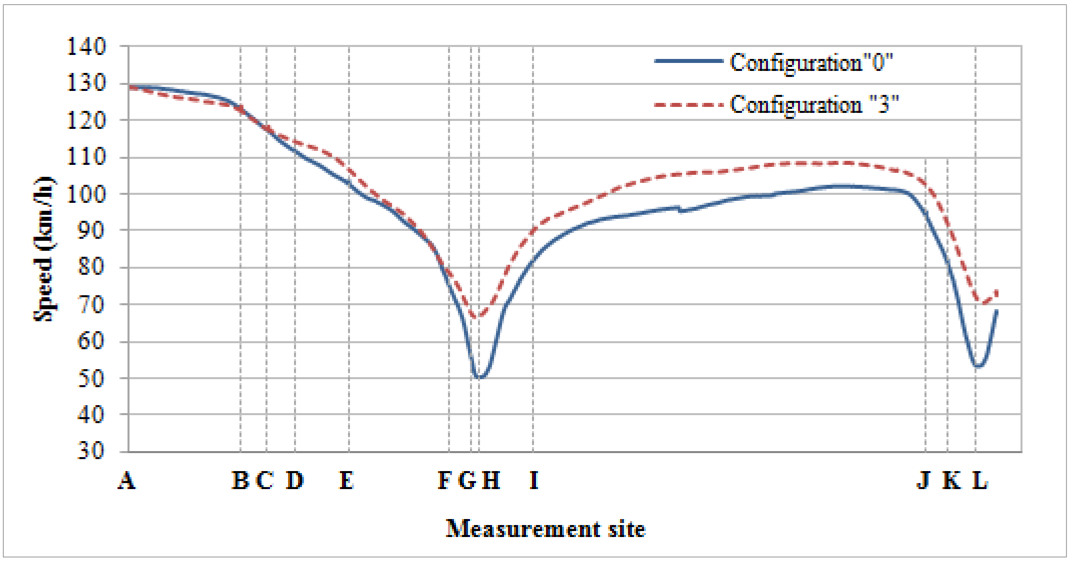



Figure 4.23: Comparison between mean speeds profiles (configurations “3” and “0”)

The analysis of the change in speed variances shows a slight decrease in the upstream-to-work-zone speed variance (+42.6%) for the configuration “3” as compared to the configuration “0” (+48.2%). However the increase of speed variance is much higher than those recorded in the configuration “1” (9.1 %). Based on this result, it is possible to conclude that the configuration “3” does not seem to provide safer conditions as compared to the configuration “1”.

### *4.3.7 The impact of perceptual treatments*

The perceptual treatments, tested within the configurations “4”, ”5”, “6” and “7”, include:

- the 1.10 m high vertical delineators placed at a distance of 3 m from each other;
- chevron alignment signs of greater dimensions than 90x90 cm standard sized chevrons;
- a 3 m tall panel with visual patterns consisting of black and yellow vertical stripes;
- a visual pattern consisting of alternating red and white stripes painted on the median barrier.

The analysis of the mean speed profile of configuration “4” shows that the use of higher vertical delineators within the transition area, in place of the flexible delineators used in the reference configuration, does not provide significant effects in reducing speeds (Figure 4.24).

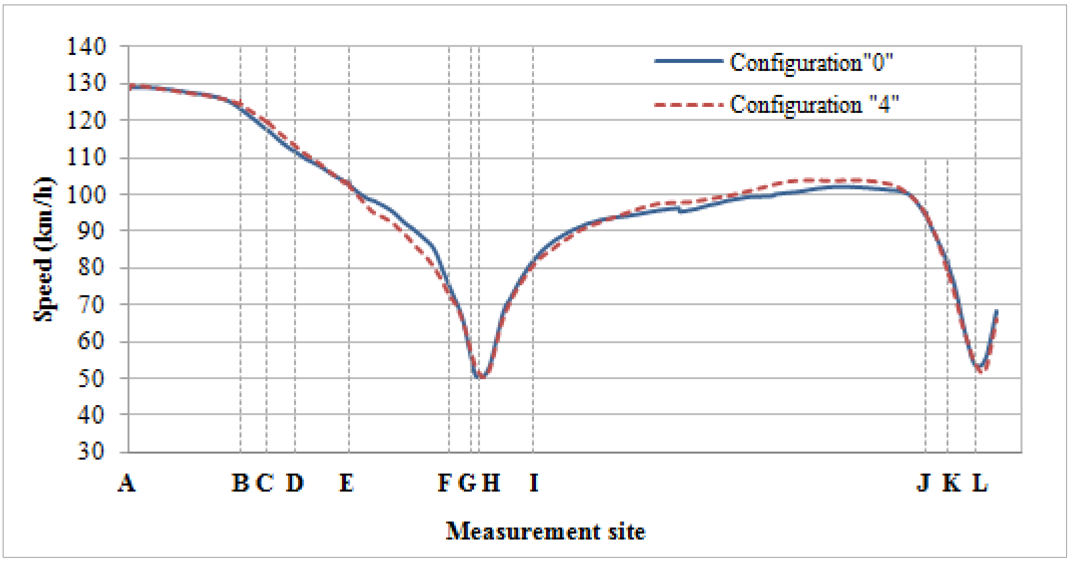



Figure 4.24: Comparison between mean speeds profiles (configurations “4” and “0”)

The presence of larger sized chevron signs in proximity of the entrance by-pass do no induce changes on drivers’ speed behaviour. However a homogenization of the speeds seems to occur in configuration “4” where a smaller increase in the upstream-to-work-zone speed variance (+16.0%) has been recorded as compared to that of configuration “0” (+48.2%).

The 3 m tall panel with visual patterns (configuration “5”) seems to provide a strong visual impact to the drivers. Indeed, examining the profiles showed in Figure 4.25 a relevant speed reduction can be noticed within the transition area, between sites E and F. Indeed, the mean speed at the “40 km/h speed limit” sign (site F) is about 5 km/h lower than that recorded in the reference configuration.

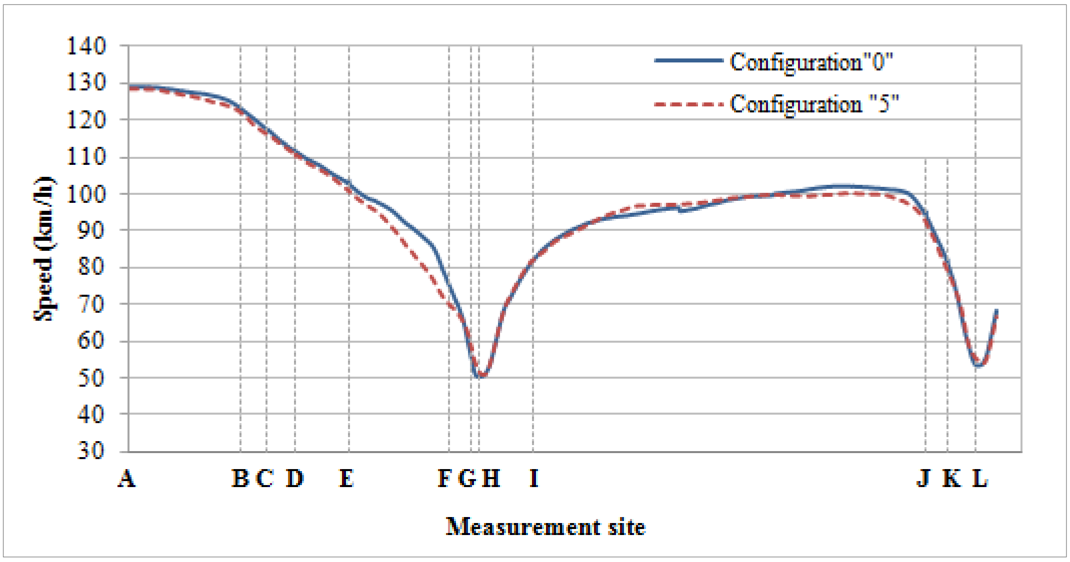



Figure 4.25: Comparison between mean speeds profiles (configurations “5” and “0”)

Furthermore, a smoother variation of the actuated decelerations has been recorded in this configuration compared to those of the reference configuration. The mean decelerations gradually increase in three different steps from 0.37 $m/s^2$ to 0.50 $m/s^2$ and then to 0.81 $m/s^2$ in the proximity of the by-pass, while in the configuration “0” they abruptly vary from 0.35 $m/s^2$ to 0.81 $m/s^2$ in two distinct phases.

The analysis of the speed variance in sites A and G shows a greater speed homogenization being the speed variance in G less than in A (-2.79 %).

The speeds held by the drivers when crossing the by-pass do not show changes as compared to the reference configuration. This result confirms that the speed within the by-pass is mostly influenced by its geometrical characteristics.

The visual pattern applied to the median barrier, coupled with taller and denser vertical delineators (configuration “7”), resulted in the largest speed reductions within the advance warning and the transition areas. A 6-7 km/h decrease in mean speed has been recorded in the segment between the site B and the site F (Figure 4.26).

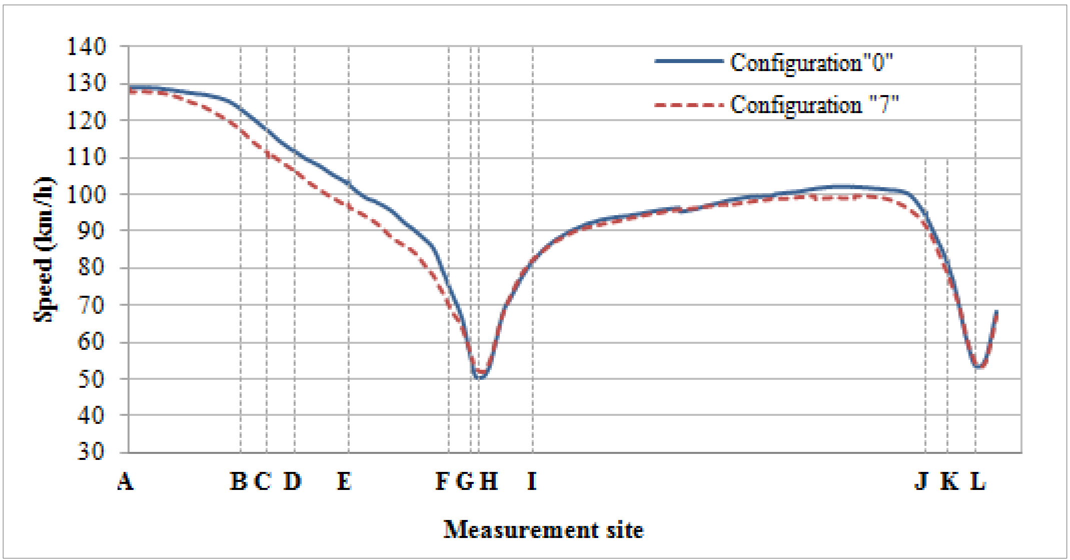



Figure 4.26: Comparison between mean speeds profiles (configurations “7” and “0”)

In this configuration a very slight variation of mean decelerations (0.34 m/s$^2$ - 0.56 m/s$^2$ - 0.66 m/s$^2$) and a reduction of the maximum deceleration value (-0.66 m/s$^2$ instead of -0.81 m/s$^2$) have been observed within the transition area.

Furthermore a greater homogenization of speeds is observed inside the work zone compared to the upstream area. Indeed, a percentage reduction in speed variance from upstream to work zone equal to 11% was recorded in this configuration.

### *4.3.8 Results of the statistical analysis*

The paired t-test was performed in order to evaluate if the mean speeds recorded at each measurement site of the reference configuration, were statistically different from those measured within the eight alternative configurations.

According to the comparison results, reported in Table 4.5 in terms of p-values, the differences between mean speeds are statistically significant at the 0.05 level in the proximity of wider median openings of the crossover (from site G to site I of configurations “1” and “3”). The visual pattern, consisting of alternating red and white stripes painted on the median barrier (configuration “5”), provides significant reductions in mean speeds within the advance warning area (from site B to site E).

Table 4.5: T-test results

| Site | Configuration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | **0_ VMS** | **1** | **2** | **3** | **4** | **5** | **6** | **7** |
| A | 0.713 | 0.822 | 0.702 | 0.860 | 0.756 | 0.904 | 0.794 | 0.852 |
| B | 0.152 | 0.359 | 0.333 | 0.817 | 0.276 | 0.739 | 0.260 | *0.067* |
| C | *0.075* | 0.718 | 0.504 | 0.748 | 0.347 | 0.698 | 0.978 | **0.033** |
| D | 0.244 | 0.549 | 0.735 | 0.458 | 0.633 | 0.630 | 0.398 | *0.094* |
| E | 0.550 | 0.829 | 0.789 | 0.377 | 0.854 | 0.213 | 0.301 | **0.042** |
| F | 0.675 | 0.985 | 0.446 | 0.241 | 0.712 | 0.264 | 0.261 | 0.206 |
| G | 0.694 | **<0.001** | 0.561 | **<0.001** | 0.468 | 0.183 | 0.721 | 0.800 |
| H | 0.763 | **<0.001** | 0.851 | **<0.001** | 0.665 | 0.754 | 0.762 | 0.460 |
| I | 0.854 | **0.004** | **0.007** | **<0.001** | 0.490 | 0.937 | 0.724 | 0.934 |
| J | 0.851 | 0.924 | 0.178 | 0.234 | 0.614 | 0.355 | 0.112 | 0.246 |
| K | 0.974 | 0.749 | 0.295 | 0.272 | 0.246 | 0.336 | 0.179 | 0.249 |
| L | 0.337 | **<0.001** | 0.224 | **<0.001** | 0.977 | 0.561 | 0.554 | 0.790 |

Note: boldface indicates statistically significant values with 5% level of significance; Italic indicates statistically significant values with 10% level of significance.

#### *4.3.9 Summary*

The comparison of the results in terms of speed variance, mean speed and deceleration values between the different configurations offers some interesting considerations.

The analysis of the percentage change of speed variances from the section upstream to the section inside the work zone showed a reduced increase in configurations “1” and “4” and a decrease in configurations “5” and “7” as compared to that measured in the reference configuration (Figure 4.17).

Thus the safe conditions achieved by increasing the speed limits and widening the median opening (configuration “1”) are also obtained by increasing the optical density of the field of view by means of perceptual countermeasures. The visual patterns applied to the panel placed and to the median barrier have been identified as the most effective measures in reducing speeds and speed variances.

Therefore, based on the theory that smaller increases in the upstream-to-work-zone speed variance cause a lower potential for crashes, the tested HF principle based configurations seem to provide the safest conditions for drivers.

The general homogenization of speeds observed in configuration “1” is accompanied by a general speeding behaviour in approaching the entrance by-pass. When the median opening is 80 m wide (configurations “1” and “3”), the mean speed values in approaching the entrance by-pass are significantly higher than those of the other configurations as a consequence of the higher manoeuvre speed allowed by the wider median opening. On the contrary, in configurations “5” and “7” the reduction in the speed variance is accompanied by a significant reduction of the mean speeds within the transition area.

The recorded mean speed values within the by-passes are very similar in all configurations with the same median opening width (Figure 4.27), regardless of the signing sequence and perceptual treatments in the advance and transition areas.

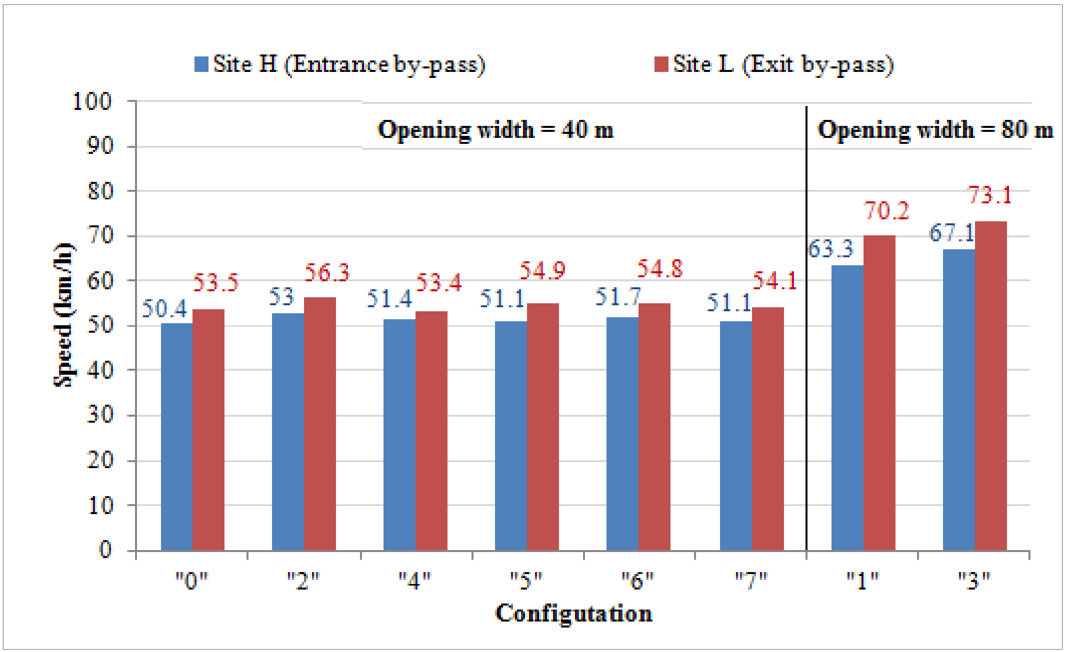



Figure 4.27: Mean speeds within the by-passes for each configuration

This confirms that the manoeuvre speed within the by-pass is influenced only by the geometrical characteristics of the median opening and not by the speeding behaviour upstream.

The maximum value of the recorded speed is attained in configuration “3” where, a greater lane width, in addition to a wider median opening, is present. This result is likely due to the fact that the lane width influences the trajectory of the travelling vehicles, leading, in case of wider lanes, to greater freedom to manoeuvre for the users in approaching the by-pass.

Finally, the analysis of the mean decelerations in the advance warning and in the transition areas allowed us to identify two distinct behaviours:

- a manoeuvre in two distinct deceleration phases (configurations “0”, “0_VMS”, “2”);
- a manoeuvre in three deceleration phases (configurations “1”, “4”, “5”, “6”, “7”): a more gradual deceleration, and therefore a more careful behaviour, is observed in approaching the by-pass.

# Chapter 5.

# Conclusions and Recommendations

The main objectives of this research were:

1. provide a better understanding of the most effective method to manage speed through work zones;
2. investigate and quantify the impact of motorway work zone layout parameters on crash occurrences;
3. investigate driver performance and behavioural changes in response to different configurations of a work zone crossover in order to identify measures leading to safer conditions for drivers.

From the literature analysis, the potential work zone parameters and the contributing causes of zone crashes were investigated. Speed and speed variance were identified as major factors in work zone fatalities, and a number of speed management strategies were investigated to address this issue.

Perceptual countermeasures appeared to be one of most promising methods, designed to reduce travel speeds by influencing speed perception, mental workload and risk perception. The use of perceptual countermeasures provides the possibility of implementing cost-effective means of mitigating speeding behaviour as well as managing variability in speeds due to their supposed “calming effects”.

The accident analysis, performed on the stationary work zones of the Italian motorway network, focused on quantifying and comparing the impact of different work zone layout configurations on the expected frequency of severe crashes (resulting in fatalities and/or injuries) through the use of the Empirical Bayes before-after method.

The findings of the analysis indicated that the overall fatal + injury expected crash frequency, during the time when a work zone is installed on a motorway segment, is about 33 % greater than the crash frequency on the same motorway segment in the pre-work zone period.

Furthermore, all layout configurations that involve a crossover resulted very critical and have the worst effects in terms of safety. The highest CMFs are observed for the layouts where traffic is partially diverted to a single lane of the opposite carriageway, requiring the driver to make a choice of which lane to use.

Work zone configurations with partial diversion of traffic to the opposite carriageway are usually needed in high-traffic sections where the full carriageway closure could generate backup queues and traffic congestion. On the other hand, the full carriageway closure with all traffic diverted in the opposite carriageway is often necessary for certain types of work and allows safer conditions for workers.

Given the high impact of this type of work zones on safety their number should be limited by combining different maintenance activities in one single work zone. The results indicated the number of lanes diverted to the opposite carriageway as a key factor for an optimal crossover design: the total diversion of traffic flow through a dual lane-crossover seems to provide safer conditions for drivers approaching the median opening as compared to the configurations with a single-lane crossover.

The purpose of the simulator experiments was to determine the safest and most effective countermeasures for the reduction of speed and speed variance within crossover work zones.

The results achieved in the driving simulation study, performed on a total sample of 42 subjects, clearly confirmed the general speeding behaviour resulting from literature findings. The drivers travel at higher speeds than those indicated by the temporary speed limits in all the work zone crossover areas and in all the configurations analyzed. Indeed, the increase of temporary speed limits did not change the mean driving speed which was significantly reduced only when perceptual treatments were included in driver's field of view. This seems to indicate that the actuated speed is not influenced by the posted speed limit but mainly by the perceived characteristics of the field of view.

The mean speed decreases significantly only within the by-passes due to their geometrical characteristics. The mean speeds recorded are approximately the same for all configurations with the same opening width, regardless of the vertical sign configurations or perceptual measures adopted within the work zone areas.

At the end of the deceleration phase, drivers perceive the by-pass as a critical section due to the particular manoeuvre to be actuated in the crossover chicane.

The increase in the median opening width from 40 m to 80 m resulted in an increase of the mean speeds between 13 and 17 km/h.

The safer driving behaviour induced by a greater homogeneity of driving speeds has been achieved by:

- adopting a wider median opening, together with higher speed limits;
- adopting perceptual countermeasures acting on the optical density of the field of view.

The second measure showed a great effectiveness not only in reducing the mean driving speed in approaching the work zone areas, but also in reducing the speed variance thus providing safer conditions. This result confirmed the optical density in the drivers' field of view as an important parameter to induce an unconscious traffic calming effect.

A possible additional safety indicator resulted in a smoother variation of the actuated deceleration in approaching the transition area: instead of actuating the deceleration in two distinct phases, as in the reference configuration, the deceleration increases more gradually in the configurations “1”, “4”, “5”,  and “7”.

The final mean deceleration recorded in approaching the by-pass was always lower in approaching the wider median opening (80 m wide instead of 40 m). This shows that the crossover manoeuvre is mainly controlled by road-vehicle interactions rather than by road-user interactions.

Among the perceptual treatments analyzed, the visual pattern applied to the median barrier (coupled with taller and denser vertical delineators installed all along the advance warning and the transition areas) led to the largest speed reductions. Such measure provides also the greatest homogenization of speeds. The panel with visual patterns located at the entrance by-pass seems to provide a strong visual impact to drivers that significantly reduce their speed within the transition area.

The installation of the VMS provides some effects on reducing speeds, although localized in the proximity of the device.

**Perspectives for future work**

The CMFs estimated in the accident analysis can provide practitioners and road operators a method to determine the expected impacts on roadway safety associated with the installation of different work zones configurations and to evaluate social costs related to possible management alternative strategies based also on the expected crash frequencies when the work zone is installed.

A survey recently conducted by Yannis et al. (2014) as a contribution to the PRACT project (PRACT, 2015) showed that CMFs for work zones for rural motorways are useful for road agencies. 86.7% of participating road agencies responded that CMFs for work zones for motorways are highly required. Despite their usefulness, 64.3% of respondents reported that there was a low availability of CMFs relating to work zones for motorways. The estimated CMFs are therefore useful to address this issue.

Future research could also examine other interesting issues not considered in the analysis. For example, CMFs related to the effects of work zone on PDO crashes and not only on fatal and injury crashes could be estimated. Furthermore the CMFs values could be estimated separately for daytime and for nighttime periods and as a function of the AADT volumes.

However, reliable data on the characteristics of the work zone at the time of crash are highly needed to perform a thorough accident analysis. Certain types of technical data, such as type of devices in use, work zone design features in place at the time of crash, cannot be effectively judged by police personnel who do not have this level of engineering expertise. Collection of this type of data by road agency personnel could be more appropriate.

Future research should also investigate alternative design features for crossovers. For example, the number of diverted lanes, together with the median opening width, is likely to be a key factor which affects traffic congestion and drivers' speeds in approaching the transition area.

The results achieved with this research might be used for the update of the Italian ministerial Decree 10 July 2002 (Ministero delle Infrastrutture e dei Trasporti, 2002).

Based on the findings of the simulator study, on-field tests should be conducted in order to validate the results for specific countermeasures. On-field tests are needed to investigate the effects of such countermeasures in real conditions as well as to estimate the expected crash reductions that may be achieved by their implementation.

Due to national regulations or road work site constraints, some parameters, such as speed limits, lanes width or geometry of the lane deviation when crossing the central reserve can't be easily tested in real work zone sites. On the other hand, the "low cost" perceptual measures, such as the visual pattern on the median barrier coupled with higher vertical delineators, could be much more easily deployed in showcase scenarios and their effectiveness could be evaluated in real site conditions.

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# Appendix A - Work Zone Configurations

### Work zone configurations for four-lane median divided motorways (two-lane carriageway)

| Stationary work zones | Description | Figure |
|---|---|---|
| Slow2 | Closure of the slow lane with traffic diverted to the overtaking lane. | Figure A.1 |
| Fast2 | Closure of the overtaking lane with traffic diverted to the slow lane. | Figure A.2 |
| Emergency2 | Closure of the emergency lane (outside paved shoulder). | Figure A.3 |
| Cross2(0+1) | Closure of the slow lane with traffic diverted to the overtaking lane; closure of the overtaking lane and total diversion of traffic to the opposite carriageway through a single-lane crossover. | Figure A.4 |
| Fast2(2) | Closure of the overtaking lane with traffic diverted to the slow and to the emergency lanes. | Figure A.5 |
| Cross2(1+1) | Closure of the slow lane with traffic diverted to the overtaking lane; partial diversion of traffic to the opposite carriageway through a single-lane crossover | Figure A.6 |

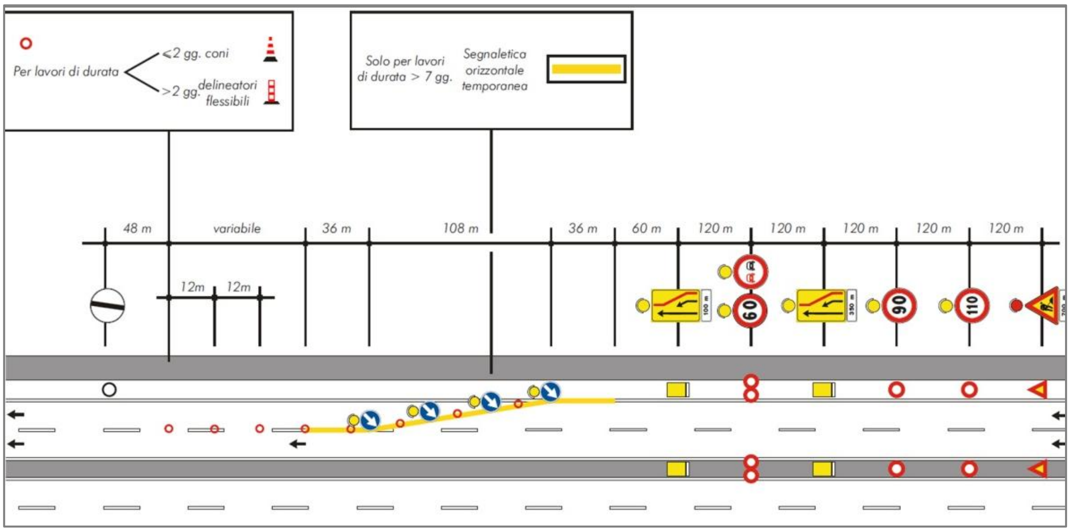


Figure A.1: Closure of the slow lane

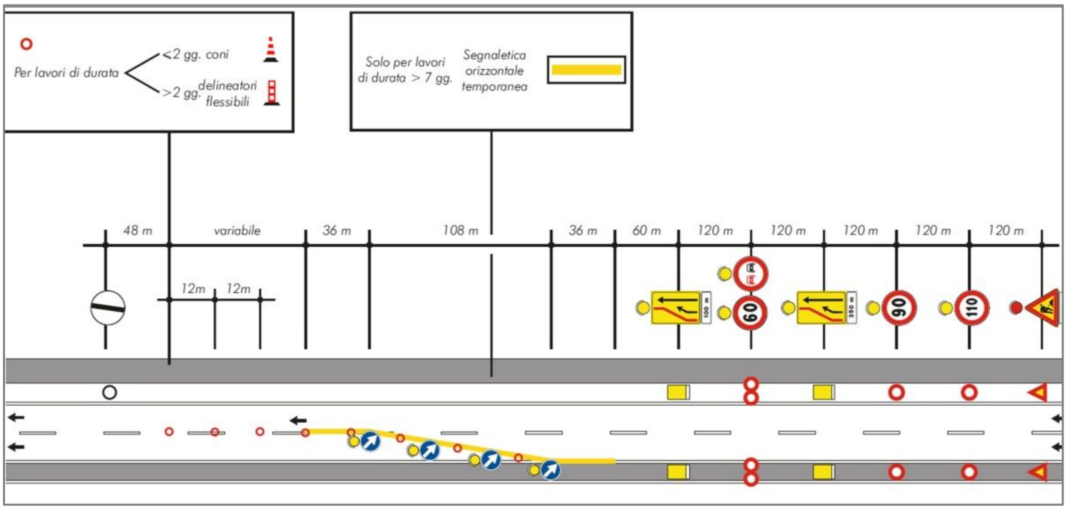


Figure A.2: Closure of the overtaking lane with traffic flowing on the slow lane

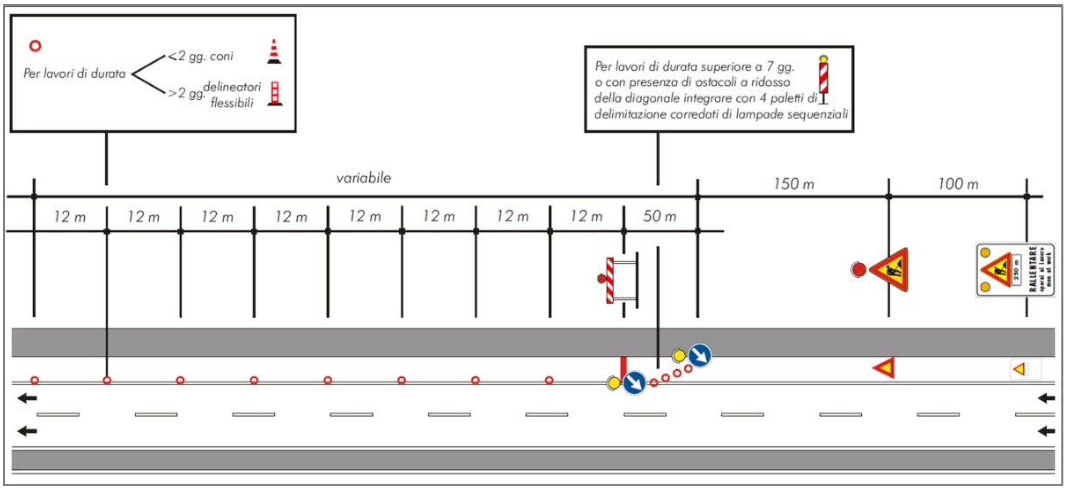


Figure A.3: Closure of the emergency lane

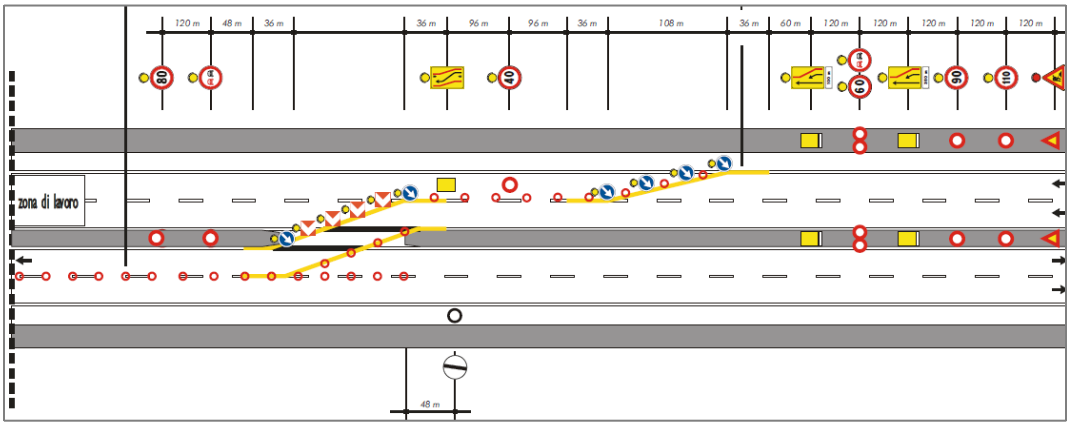


Figure A.4: Total diversion of the flow on a single lane of the opposite carriageway

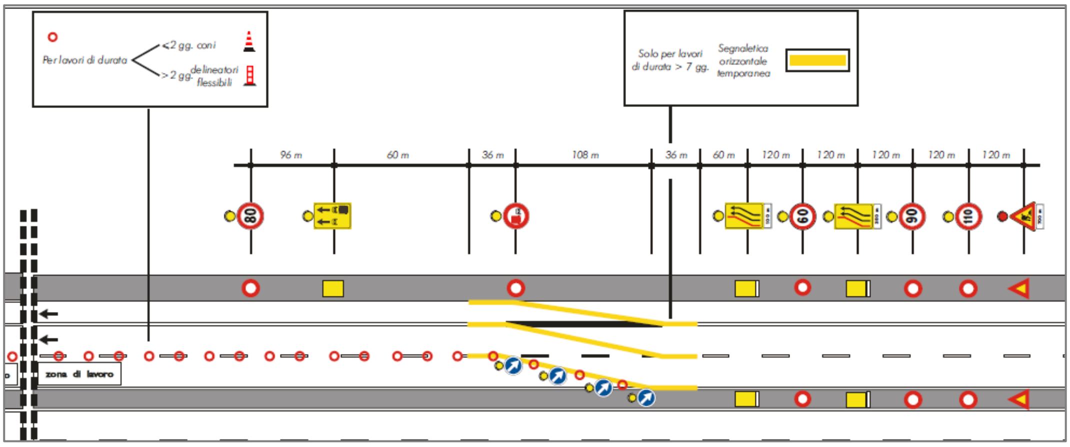


Figure A.5: Closure of the overtaking lane with traffic still flowing on two lanes (on the slow lane and on the emergency lane)

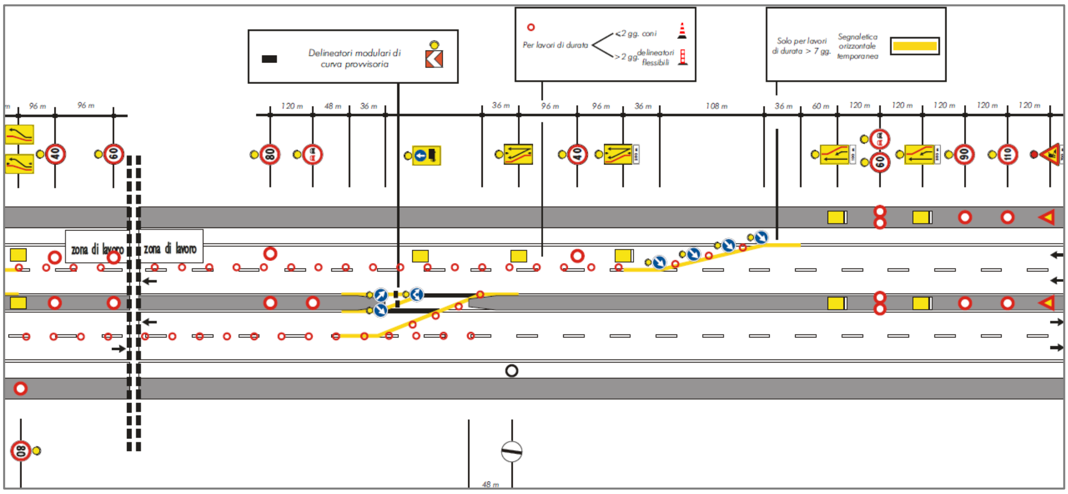


Figure A.6: Partial diversion of the flow with a single lane for the traffic not diverted and a single lane for the traffic diverted in the opposite carriageway

## Work zone configurations for three lanes carriageway

| Stationary work zones | Description | Figure |
|---|---|---|
| Slow3 | Closure of the slow lane with traffic diverted to the middle lane. | Figure A.7 |
| Emergency3 | Closure of the emergency lane (outside paved shoulder). | Figure A.8 |
| Fast3 | Closure of the overtaking lane with traffic diverted to the middle lane. | Figure A.9 |
| Slow&Middle3 | Closure of the slow lane with traffic diverted to the middle lane; closure of the middle lane with traffic diverted to the overtaking lane. | Figure A.10 |
| Middle&Fast3 | Closure of the overtaking lane with traffic diverted to the middle lane; closure of the middle lane with traffic diverted to the slow lane. | Figure A.11 |
| Cross3(0+1) | Closure of the slow lane with traffic diverted to the middle lane; closure of the middle lane with traffic diverted to the overtaking lane; closure of the overtaking lane and total diversion of traffic to the opposite carriageway through a single-lane crossover. | Figure A.12 |
| Cross3(1+1) | Closure of the overtaking lane with traffic diverted to the middle lane; closure of the middle lane and partial diversion of traffic to the slow lane and to the opposite carriageway through a single-lane crossover | Figure A.13 |
| Middle&Fast3(2) | Closure of the overtaking lane with traffic diverted to the middle lane; closure of the middle lane with traffic diverted to the slow lane and to the emergency lane. | Figure A.14 |
| Fast3(3) | Closure of the overtaking lane with traffic diverted to the middle lane, to the slow lane and to the emergency lane. | Figure A.15 |
| Cross3(0+2) | Closure of the slow lane with traffic diverted to the middle lane; closure of the carriageway and total diversion of traffic to the opposite side through a dual-lane crossover. | Figure A.16 |

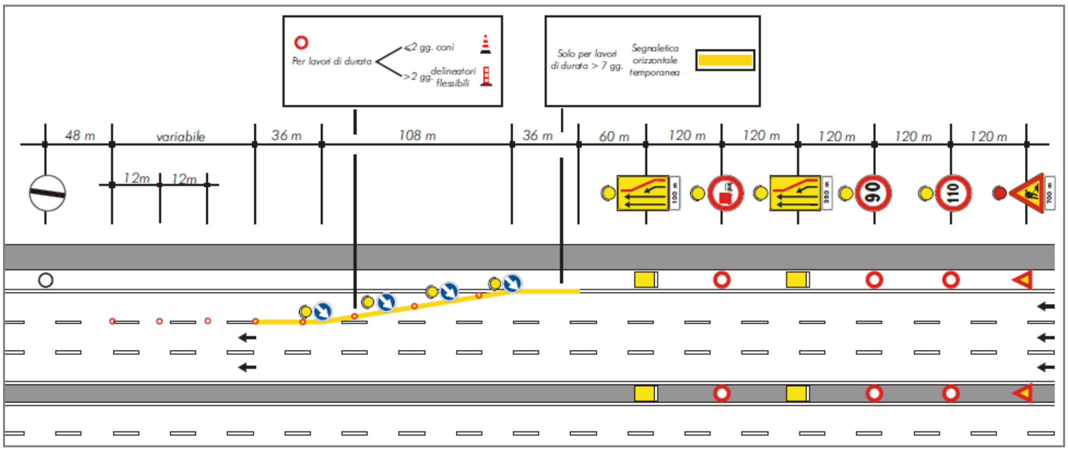


Figure A.7: Closure of the slow lane

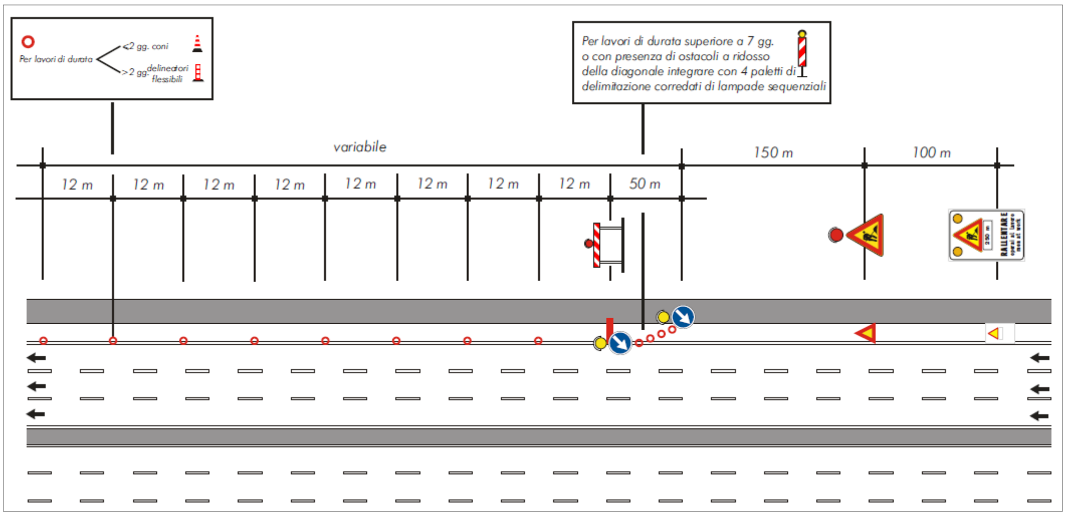


Figure A.8: Closure of the emergency lane

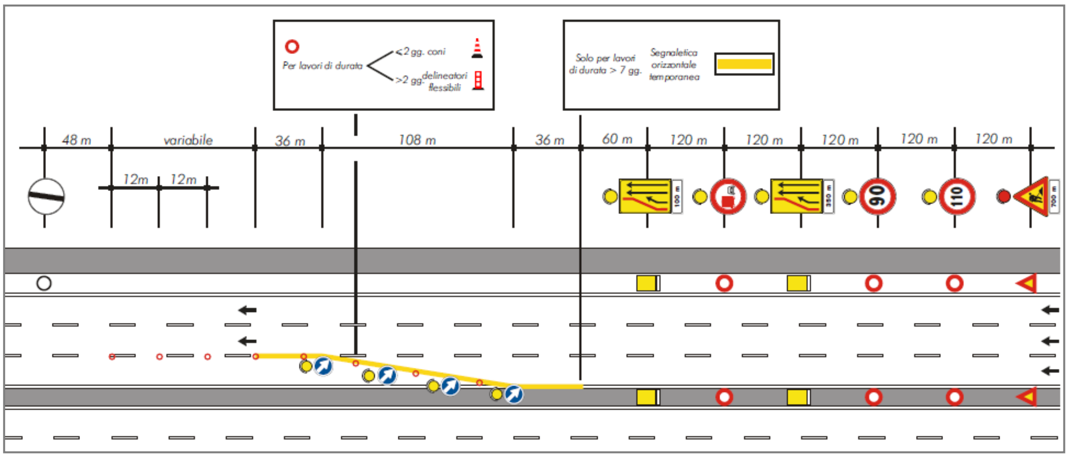


Figure A.9: Closure of the overtaking lane

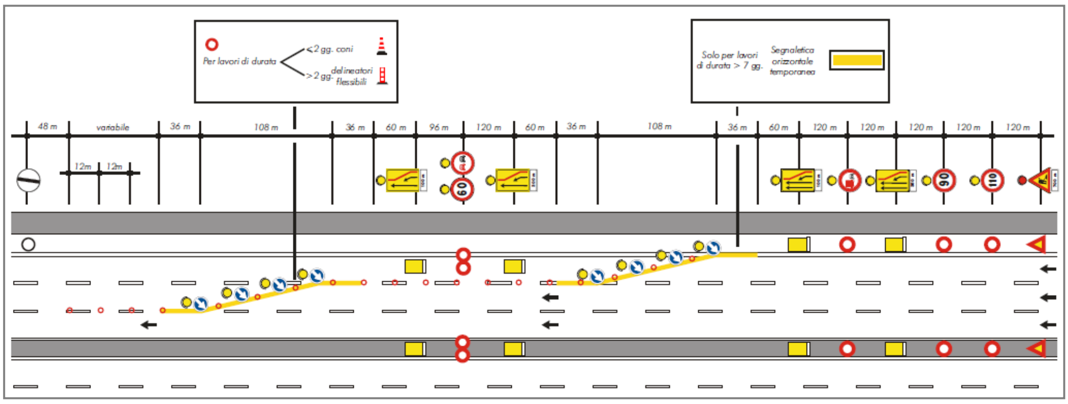


Figure A.10: Closure of the slow lane and middle lane

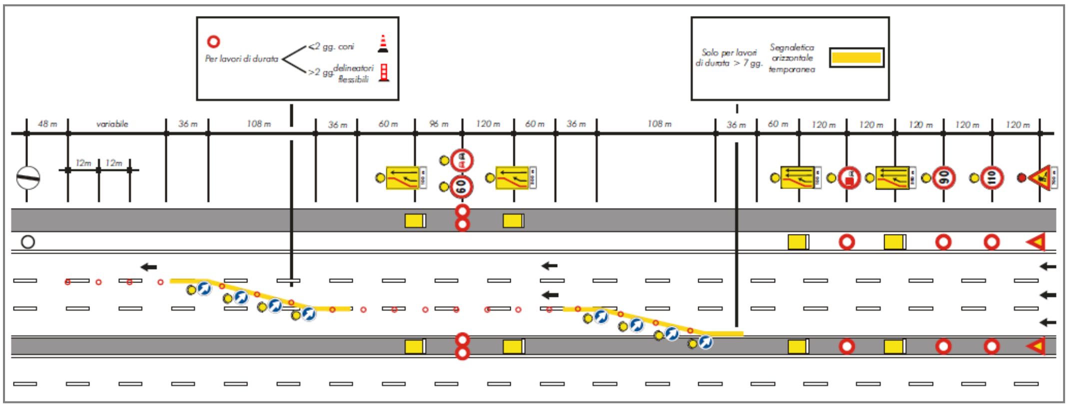


Figure A.11: Closure of the overtaking lane and middle lane

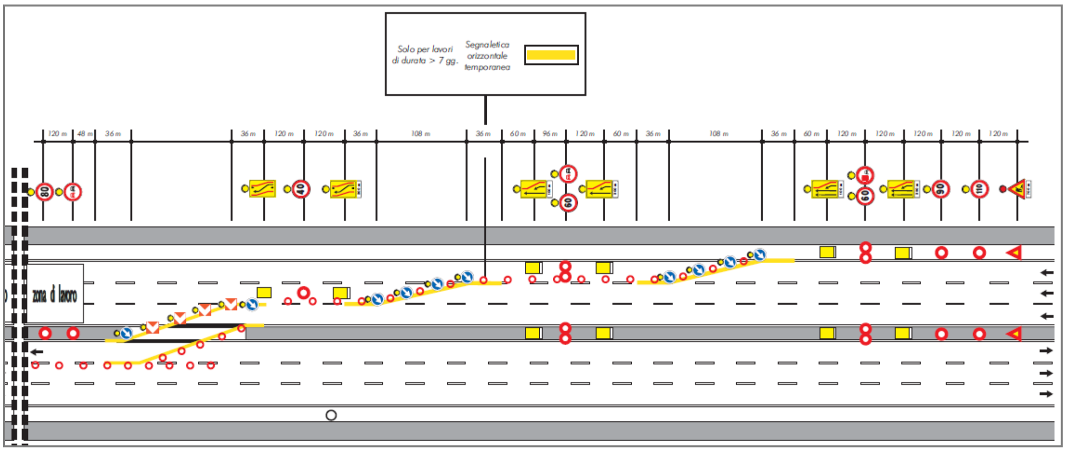


Figure A.12: Total diversion of the flow on a single lane of the opposite carriageway

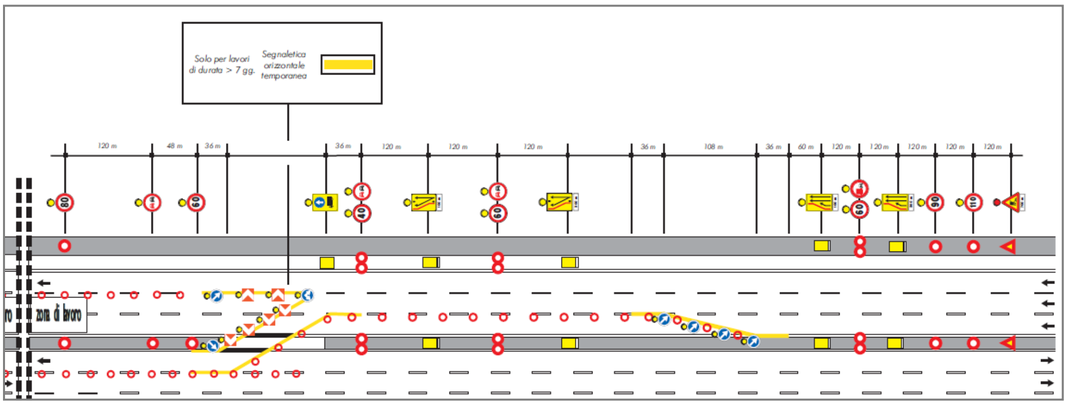


Figure A.13: Partial diversion of the flow with a single lane for the traffic not diverted and a single lane for the traffic diverted in the opposite carriageway

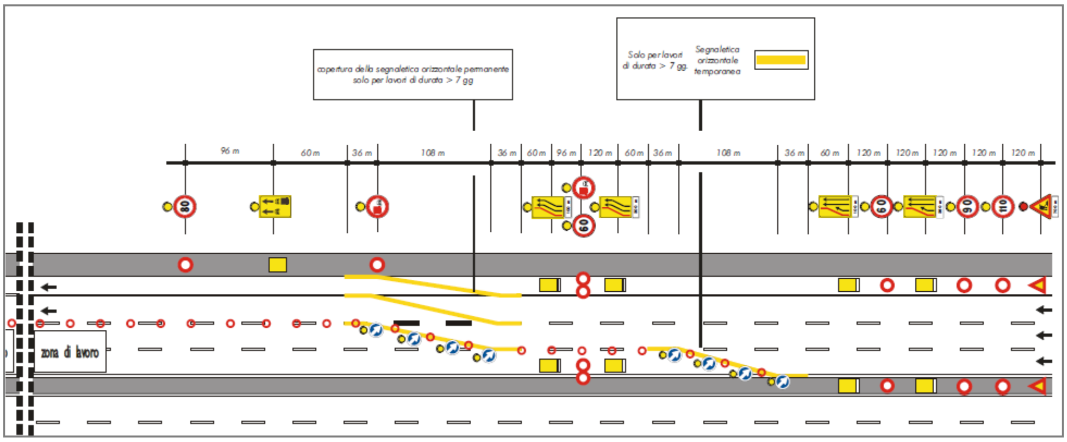

Figure A.14: Closure of the overtaking lane and middle with traffic flowing on two lanes (on the slow and emergency lanes)

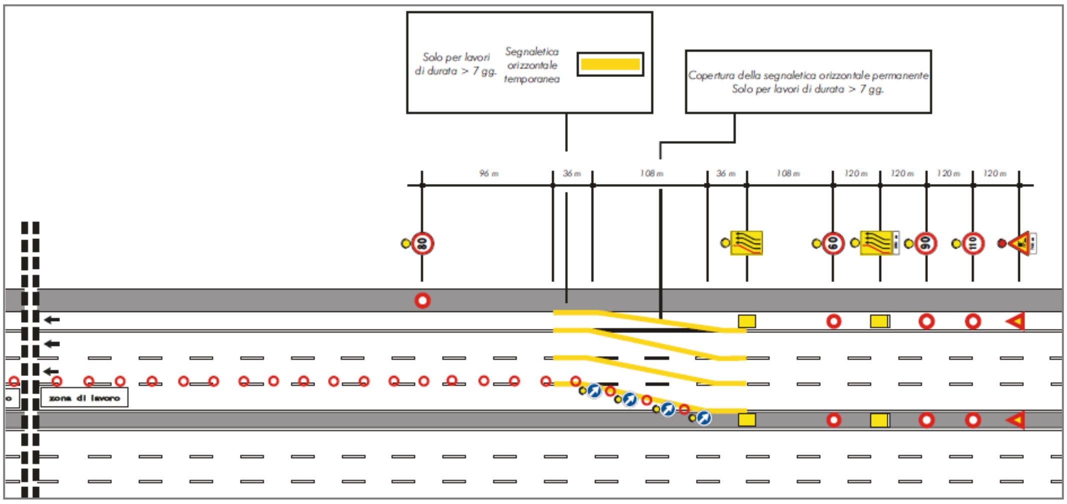


Figure A.15: Closure of the overtaking lane with traffic still flowing on three lanes (on the middle, slow and emergency lanes)

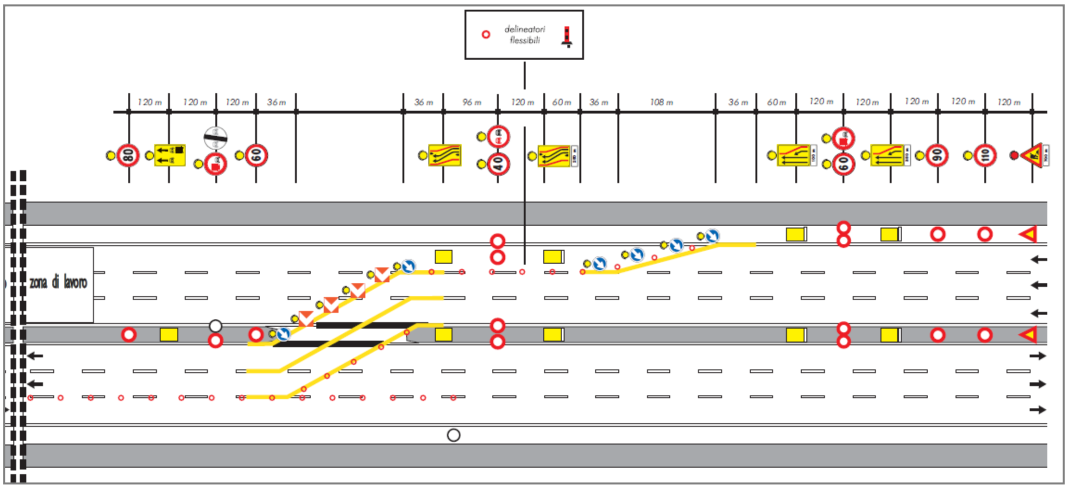


Figure A.16: Total diversion of the flow on two lanes of the opposite carriageway

# Appendix B - Pearson's chi-square Test

Pearson chi-square test method is known as one of the most popular test methods for independence between two sets of variables. In order to test the independence of two variables A and B having a and b values respectively, the test statistic is a chi-square random variable ( $\chi^2$ ) defined by the following equation:

$$\chi^2 = \sum_{j=1}^{b}\sum_{i=1}^{a}\frac{(O_{ij} - E_{ij})^2}{E_{ij}}$$

Where:

- $O_{ij}$ is the observed frequency with attributes $A_i$ and $B_j$;
- $E_{ij}$ is the expected frequency with attributes $A_i$ and $B_j$.

The expected frequency counts ( $E_{ij}$ ), computed separately for each level of one categorical variable, are calculated as follows:

$$E_{ij} = n \cdot (O_{i.} / n) \cdot (O_{.j} / n)$$

Where:

$$O_{.j} = \sum_{i=1}^{a} O_{ij}$$

$$O_{i.} = \sum_{j=1}^{b} O_{ij}$$

- n is the total sample size.

The null hypothesis $H_0$ and the alternative hypothesis $H_a$, formulated to test the independence between A and B, are:

- $H_0$: Variable A and Variable B are independent;
- $H_a$: Variable A and Variable B are not independent.

The test was performer with a significance level of 0.05. Therefore when the p-values, associated with chi-squared distribution with *df* degrees of freedom (*df = (a - 1) (b - 1)*) was less than 0.05 then the null hypothesis was rejected and the two variables were considered related to each other.

SPSS software package was used to perform the statistical analysis on crashes.

# Appendix C - Post-training questionnaire

Laboratorio per la Sicurezza e l'Infortunistica Stradale

## QUESTIONARIO POST FASE DI TRAINING PER LA SPERIMENTAZIONE CON SIMULATORE DI GUIDA *(Post-Training Questionnaire)*

I dati raccolti in questo modulo saranno inseriti in forma anonima nel database del LaSIS e saranno utilizzati per fini statistici nell'ambito del progetto ASAP

**Codice Utente *(User Code)*: ID ….._2012_ASAP**

*A) VALUTAZIONE DEL DISAGIO PERCEPITO DURANTE LA SIMULAZIONE DI GUIDA*
***(Evaluation of perceived driving discomfort)***

1. **Ha provato senso di malessere, disagio durante la guida al simulatore?**
*(Did you experience symptoms of simulator sickness?)*

☐ SI *(Yes)* ☐ NO *(No)*

2. **Se sì, descriva il tipo di malessere:**
*(if so, describe your symptoms)*

☐ Nausea *(Nausea)*
☐ Mal di testa *(Headache)*
☐ Giramento di testa *(Dizziness)*
☐ Stordimento *(Daze)*
☐ Affaticamento degli occhi *(Eye strain)*
☐ Altro *(other)* (specify)……………………………………………………………………

3. **Ha interrotto la sessione di guida a causa di un malessere?**
*(Did you stop your driving session due to symptoms of simulator sickness?)*

☐ SI *(Yes)* ☐ NO *(No)*

4. Se sì, identificare il tipo di malessere *(if so, describe your symptoms)* : ……………………………

## B) VALUTAZIONE DEL REALISMO DELLA SIMULAZONE DI GUIDA *(Evaluation of perceived realism in driving simulation)*

**1.** Esprima un giudizio da 1 a 10 sulla fedeltà relativa ai seguenti elementi:
*(Please rate from 1 to 10 on the realism of each the following items)*

- Velocità *(speed)* ………………
- Accelerazione *(acceleration)* ………………
- Frenata *(braking)* ………………
- Cambio *(gearbox)* ………………
- Volante *(steering wheel)* ………………
- Percezione del tracciato *(trackperception)* ………………
- Percezione dell'ambiente circostante *(perception of the sorrounding enviroment)* ………………
- Percezione del rumore del motore *(engine sound perception)* ………………

2. È riuscito a guidare il simulatore:
*(How did you drive with the simulator?)*

- ☐ Senza difficoltà *(without any difficulty)*
- ☐ Con qualche difficoltà *(with some difficulty)*
- ☐ Con molte difficoltà *(with great difficulty)*

3. Come valuta il comportamento di guida mantenuto durante la simulazione:
*(How do you evaluate your driving behaviour with the simulator?)*

- ☐ Simile al comportamento durante la guida reale *(similar to real driving)*
- ☐ Intermedio al comportamento durante la guida reale *(quite different from real driving)*
- ☐ Diverso dal comportamento durante la guida reale *(totally different from real driving)*